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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 1 MARCH 1938

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 1 March 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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FINANCING OF LOCAL ROADS AND STREETS

REPORT OF DEPARTMENT OF HIGHWAY FINANCE, HIGHWAY RESEARCH BOARD, NATIONAL RESEARCH COUNCIL[1]

Presented by THOS. H. MACDONALD, Chief, Bureau of Public Roads, Chairman

CONSIDERATION of present and past methods of highway financial administration reveals slow progress toward a rational spending program. Demands for increased highway appropriations have in many cases diverted attention from the need for wiser spending of what we have and more efficient managing of what we spend. Design, construction, and maintenance standards have kept reasonable pace with modern transport tempo, but policies of administration and finance remain essentially horse-drawn.

Highway tax distribution and the administrative difficulties involved have been examined with particular reference to local application of State funds for highway purposes. Last year more than a quarter of a billion dollars in State gasoline taxes and registration fees were set aside for roads and streets not on the State highway systems. This money was 25 percent of total motor-vehicle tax collections for 1936. The large part of highway-user taxes so distributed is an index of the need for studying methods of allocating such funds to local governments, for establishing an economic basis for shared taxes and State aid, and for inquiring into the uses to which these funds are now applied, the degree of financial control retained by the States, and the fiscal and managerial pitfalls into which both State and local governments spend their way.

Vehicle taxes for local roads.—That highway users should be charged in accordance with their utilization of highway facilities is the generally accepted theory upon which the gasoline tax and registration fee are established. It appears to follow therefore that the distribution of such taxes to various parts of the highway system should reflect the relative traffic volumes which they carry.

In the period of rapid highway expansion which paralleled the growth of motor-vehicle travel, the theory that those who used the roads should pay for them was generally conceded, but financial pressure created by the need for a new system of main roads made it neither possible nor desirable to adopt the corollary that funds should be spent with exact regard to their origin. With the progress of a primary system of highways which such concentrated finance made possible, however, there originated in both counties and municipalities a demand that some part of State tax collections be returned for local roads and streets. Today the wide range in relative proportions of funds made available to local governments suggests no more scientific consideration than the loudness of these demands. In 1936, 3 States returned more than half of total highway user imposts to local units of government, 11 over one-third of such collections, and 5 States made no allotments whatever. Local roads in 1 State received 24 million dollars in State taxes, while in each of 10 other States less than a million dollars were distributed for highways in local jurisdictions.

Tax distribution laws.[2]—State laws governing the amount and basis of gasoline tax and registration fee distribution comprise a legal labyrinth which varies in complexity from State to State. Two considerations are involved: Determination of the total which shall be distributed by the State; and the division of this sum among the various local units. The total share going to local roads is generally expressed as a percentage of collections, a specific part of each tax levied, or a predetermined flat sum. The allocation to each local unit may then be made according to the population, area, assessed valuation, road mileage, or on the basis of vehicle registrations or tax collections. In the case of the registration fee, however, shares are often retained by each separate local unit at the time of collection, either as a fixed amount per registration or a percentage of total receipts.

Although the total amount of motor-vehicle taxes granted for local road purposes may have no relation to traffic needs originating on these systems, in a large number of States registration fees are allocated among the separate units with a regard for relative traffic potentialities. Thus Arizona counties retain 50 cents for each original registration, while in Alabama 20 percent of total receipts from this source are used in the counties where the taxpayers reside. In the case of the gasoline tax, however, not only does the original sum granted by the State have little bearing upon traffic volume and intensity, but also the allocations among individual local units are generally based upon formulas which are untenable. Alabama, for example, distributes 3 cents of a 6-cent tax equally among its 67 counties, while New York counties receive 20 percent of collections according to the road mileage of each county. In Tennessee 1 cent of the gas tax is distributed to the counties equally, ½ cent on county areas, and ½ cent according to county populations.

STANDARDS NEEDED TO GOVERN ALLOCATION OF ROAD FUNDS

When money is distributed equally among local road units which vary in size and stage of development, or on the basis of land areas and road mileage which bear no relation to traffic conditions, there is little chance that distribution will be economically justifiable. Only by chance will highway income be in reasonable balance with the demand for funds. Even population and assessed valuation may be poor indices of the proper share of taxes required by local governments for transport facilities. Questionable practices of tax allocation accordingly help to make possible such variations in road expenditures as found in North Carolina before the State assumed control of all rural roads. The annual road expenditure in one county was $14 per mile, while in another it was $688. Similar conditions were found in Iowa in 1933 by a study of the Brookings Institution, which revealed that if State funds were distributed on the basis of some defensible index such as traffic or vehicle registrations (instead of area) allotments would have been reduced considerably in 75 percent of the counties.

In general the conclusion may be drawn that present methods of State fund allocations to local roads and streets are no less heterogeneous and unscientific than

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[1] Presented at the Seventeenth Annual Meeting of the Highway Research Board, December 2, 1937.

[2] Details by States are given in the appendix, table A.

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are the rates and bases of the taxes through which these funds are raised.

Economics of user tax distribution.—The question of what share of State motor-vehicle taxes should rightly be allocated to roads and streets other than on the primary system involves fundamental concepts of highway economics. The purpose of roadbuilding is to provide for adequate traffic facilities at the lowest possible cost, including both road costs and vehicle operating costs. In spending for the highway program, therefore, funds must be allocated to those parts of the transportation system where improvements will bring about the greatest reduction in total cost and the greatest utility in adequate service.

Since limited funds do not permit simultaneous betterment of all roads, the element of time is of great moment in an economic distribution of vehicle taxes. If funds were returned to local roads and streets in the amounts generated thereon, prior to adequate development of a system of main highways, the higher cost of transportation for the many vehicles on congested primary routes would far outbalance the reduction in operating costs on the local roads. Also, whereas two road systems may carry equal amounts of traffic, expressed in vehicle miles or gasoline tax receipts, yet the needs of either depend largely on the type and distribution of this travel: Whether highway utilization has been intensive, as on heavily trafficked main roads, or extensive, as the dispersed use of a large network of local rural roads. It must also be known in what ratio heavy trucks and busses or pleasure vehicles have accounted for traffic volumes. Furthermore it is important to recognize the integration of motor travel on the various road systems, and the fact that it is the entire trip which must be made at lowest cost, as well as the entire motoring population which must be considered in the computation of total costs for the entire highway system.

The aspect of the principal routes as revenue producers is sound in principle. So large a percentage of the actual use of these is recreational in character that the potential increase by reason of wholly adequate facilities should be self-evident—not only this, but the competitive nature of recreational offerings. The highways must compete with other classes of recreational inducements. In the business of tourist traffic one route becomes competitive with other routes, region with region, and even State with State. The impact of the degree of adequacy of major highways has large effects upon both private and public income. The financial support for local road improvements depends to great extent upon the excess earning capacity of the main roads, which in turn is dependent upon the attraction of potential traffic resulting from the offering of satisfactory facilities.

Broader understanding of the purpose of a highway transportation system, viewed as an entity, will demonstrate the importance of such concepts as priority and intensity of use, rather than integrated vehicle mileage alone, as standards by which tax allocations must be measured and financial policies adopted.

Trend in State tax distribution.—Of the total collections of State motor-vehicle taxes in 1927, 73.1 percent were used for State highway purposes and 22 percent for local roads and streets. By 1936 the percentage of user taxes spent on State roads had decreased to 55.2 percent, while local road allocations increased slightly to 25.1 percent. During this 10-year period, however, total vehicle taxes increased 90 percent, so that the reduced State highway share still represented a 4 percent dollar increase, and the 3.1 percent rise in the local road allotment was an actual 115 percent dollar increase. These figures are shown in table 1.

TABLE 1.—*Distribution of motor-vehicle taxes for highways, 1927 and 1936* [1]

Year	Total vehicle taxes collected	Amount for State highways	Percent	Amount for local roads and streets	Percent
1927	$560, 027, 983	$409, 596, 885	73. 1	$123, 176, 360	22. 0
1936	1, 057, 995, 000	583, 616, 000	55. 2	265, 496, 000	25. 1
Percentage change 1927–36	+90	+42	−17. 9	+115	+3. 1

[1] Detailed tabulation appears in the appendix, table B.

It will be noted that whereas in 1927 vehicle funds available for highway purposes were 95.1 percent of the total, in 1936 only 80.3 percent of tax collections were used for highways. This increasing use of road funds for other purposes appears to have hit hardest the State highway systems, though hidden and unreported diversions by local units of government make impossible any definite statement on this subject.

CONTROL OF LOCAL ROADS BEING TRANSFERRED TO STATES

There has been more widespread recognition in the past decade of the right of subordinate units of government to share in State taxes. For whereas 20 States distributed gasoline taxes to local roads and streets in 1927, in 1936 there were 36 States making such allotments. Registration fees were used for local roads by 27 States in 1927 and by 32 States in 1936.

City streets.—Because funds allotted to counties in many States may be used within municipalities, and because such expenditures are not always reported separately, it has not been possible to determine accurately the amount of State money spent on city streets. Accordingly these sums have been included with local road apportionments, and expenditures on urban extensions of State systems have been included in State highway disbursements where it has been possible to segregate them from other local road and street funds. The best figure obtainable for State money spent on city streets is $31,468,000, compiled by the United States Bureau of Public Roads for 1936. Eleven States report such expenditures.

Administrative set-ups.—Highway administrative agencies in the United States include the States, counties, towns, and townships, incorporated cities and villages, and miscellaneous local divisions of government. In each State the size, type, and number of such agencies in operation, and the relation or lack of relation among them, differ widely.

In 4 States all rural roads are administered by the State highway departments, while 26 States [3] have State and County organizations; 6 have State and township systems; and 12 have three systems: State, county, and township. In addition to these rural systems, all States contain municipal organizations which have charge of urban streets, and half the States have further independent or semi-independent divisions within the county, such as commissioners' districts and special assessment districts, both rural and urban.

In most States there is neither control by the State over the spending of funds allocated to lesser governmental units, nor is there cooperation between the State and local highway organizations. Where laws designate that the State shall approve county construction programs financed with the assistance of State funds, such approval is not uniformly followed by adequate supervision of the actual work. Where counties are invited to seek the aid and advice of the State, in practice the results are far from reassuring.

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[3] Including the State of Washington, although 2 of its counties still contain township units.

Trend toward centralized administration.—At the close of 1930 there were 324,496 miles of highways under State control. By the end of 1936 State-controlled mileage had increased to 533,144 miles, a 64.3 percent addition in 6 years. Such has been the progress of a movement toward centralized highway administration which began in North Carolina in 1931. By assuming control over the State's 46,800 miles of county roads, North Carolina was the first to consolidate its entire rural highway system under the State highway department.

It was not long, however, before complete centralization was adopted in West Virginia, Virginia,[4] and Delaware. In Maryland 20 out of 23 counties have turned over their roads for maintenance by the State, while a program of consolidation under way in Pennsylvania has resulted in State participation in the maintenance of 46,000 miles of township secondary roads. On January 1, 1938, a total of 2,574 miles of Pennsylvania roads in townships, boroughs, and cities will be absorbed by the State. Popularity of the road consolidation program since 1931 may be judged by figures in table 2, which show highway transfers to the State highway departments.

Twenty-six separate transfers have been made in the 6-year period 1931–36, involving 21 States and nearly 172,000 miles. It is of interest that last year 10 States were involved in such transfers, or twice the number in any previous year.

TABLE 2.—*Transfers of local road mileage to the State highway departments* [1]

Year	Number of States	Mileage involved
1931	3	73, 651
1932	1	37, 028
1933	3	37, 744
1934	5	7, 190
1935	4	5, 623
1936	10	10, 696
Total	[2] 21	171, 932

[1] A detailed tabulation appears in the appendix, table C.
[2] Several States effected more than 1 consolidation.

Further consolidations have been effected among the lesser units of government in the assumption of township road responsibilities by county highway organizations. It is generally conceded that the township, which in most cases contains an area of 36 square miles or less, has no place in efficient highway administration, and in the past 7 years four States have done away with these ineffective highway administrative agencies and adopted a so-called county-unit form of highway organization. With this type of administration all roads within the county and not a part of the State system are operated as a unit, with locally collected taxes in townships and districts being spent by the central county administration without regard to township or district lines. This county unit plan makes possible more economical use of road machinery, a broader tax basis, cooperation and planning, economy in maintenance operations, quantity purchasing, and necessitates the budgeting of funds and the keeping of cost records. When Michigan recently completed the transfer of 60,000 miles of township roads to county-unit control, there were eliminated 1,376 small administrative units.

STATE ADMINISTRATION OF ROADS MORE ECONOMICAL THAN LOCAL ADMINISTRATION

Causes of consolidations.—The immediate cause leading to centralization of road administration in North Carolina appears to have been the public desire, accentuated by economic depression, to escape from county property-tax levies. It was proposed that the State assume all future highway financial requirements, with the aid of a 1-cent increase in the State gasoline tax, except that the counties should continue payment for the servicing of highway obligations previously incurred. The shift of financial responsibility, then, was from property to motor vehicles and from local governments to the State.

This centralization plan, however, suggests something more than a temporary relief measure. For it is doubtful that the counties would have acceded to such surrender of autonomy had the past record of county highway administration proved efficient and economical. That such terms could not be applied to a majority of North Carolina counties was evident from the conditions which the State found in existence upon taking over local road affairs. Instead of 67,000 miles of roads listed by the counties only 45,000 miles could be found, despite the fact that 2,590 miles had not been accounted for in the original figure. Maintenance varied from satisfactory standards to hopeless inadequacy, and maintenance records in many counties did not exist. Some counties were found oversupplied with machinery, others practically destitute, and in nearly all cases machines were either obsolete or badly in need of repair. Such causes as these, rather than temporary tax relief, are thought to have been fundamental in the trend toward State assumption of local roads. That the trend has not slackened with return to more normal economic conditions may have a bearing upon this point.

Property taxes for roads.—Whatever is to be said for or against State centralization of highways, the concomitant policy of relieving property of its share in supporting the highway does not conform with the generally accepted theory of highway economics: That costs should be paid in accordance with service rendered. The shifting of road administration from local to State control involves no alteration in the principle that highways serve other functions than those directly relating to motor vehicles. In an equitable allocation of highway costs, rational payments for land service are rightly chargeable to the land which is served. Property levies are an essential part of highway income, and their elimination may not only deter a proper development of highway facilities, but may also constitute an unfair burden upon the motorist.

A second criticism of policy in connection with highway centralization concerns the tendency of the State to neglect its first responsibility of preserving the integrity of the primary road investment and of providing necessary extensions. A shift in administration does not relieve the State of obligations previously assumed, and the requirements of the main road system must be recognized prior to further tax allocations.

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[4] Except 3 counties which have elected to retain control of local roads.

A large element of overriding the recommendations and warnings of the State highway departments has characterized the adoption of State policies throwing the cost burden of additional large mileages upon the incomes from user taxes available to the departments and usually inadequate for the requirements of the existing major highway systems.

Criticism of small administrative units.—It is self-apparent that many small roadbuilding entities now in operation are outworn relics of the dependence of transportation upon the horse: That both the time and distance of travel upon which their limits were fashioned have been reduced to negligible importance. Administrative scope has expanded, and this fact must be recognized by eliminating the multiplicity of highway organizations of minor units of government which make impossible the operation of highways as a coordinated system. A small unit is generally unable to afford proper engineering personnel, its staff may be subject to frequent changes because of elections, and in general undue emphasis is likely to be placed upon political rather than technical considerations.

Short radii of operation make the use of modern road machinery uneconomical through excessive overhead and numerous duplications, while small purchases of supplies and materials impose penalties of higher unit prices. Variations among the jurisdictions in area, population, taxable valuation, road mileage, topography, climate, vehicles registered, and traffic volumes, may make possible the extension of road facilities beyond traffic requirements in one county, while a neighboring unit may be financially unable to provide the taxpayer with a lasting return for the money he pays for satisfactory highway services. Budgeting, accounting, debt control, and planning are generally beyond the pale of local road administration; while lack of continuous maintenance, the use of force account methods, and incompetently controlled spending of funds collected outside the local jurisdiction are weaknesses generally in evidence.

OPTIMUM SIZE OF HIGHWAY ADMINISTRATIVE UNIT DISCUSSED

Variations among counties and States.—In most discussion relating to the merits or demerits of centralized government it is claimed on the one hand that the county is "too small" to effect a proper highway administration, and on the other that the State is "too large." Either statement implies that counties and States are essentially homogeneous, and that there exists a standard-size government unit most applicable to proper highway management. Yet neither counties nor States are homogeneous units. Counties may differ in area from the 25 square miles of Arlington County, Virginia, to San Bernardino's 20,175 square miles in California. This latter county is larger than the three States of New Jersey, Delaware, and Maryland combined.

In population, variations are even more pronounced, Loving County, California, for example, having but 195 residents compared with 4 million persons living in Cook County, Illinois. As regards the States, the largest area is 250 times that of the smallest, while populations vary in the ratio of 138 to 1. Nine States have more than 100,000 miles of highways (Texas has over 200,000) while six have less than 15,000. The fact that a county may be larger than the State of Delaware, in which State centralization of highways is in effect, presents the possibility that the State may actually be "too small" and the county "too large."

Consideration of the county as a highway administrative unit must take into account the two different general types of county, the rural, and the urban. It is the rural county which is so often unadapted to the performance of highway functions because of the limitations of its resources and the lack of sufficient highway activity to permit large-scale operations, either intensive or extensive. The urban county which contains a large city and considerable traffic and population, however, is, by reason of its wealth, responsibilities, and intensive road needs, a logical highway administrative unit. Such urban counties nevertheless are handicapped in their function of improving highways by reason of the fact that they are usually part of a larger metropolitan area embracing more than one county, as well as lesser jurisdictions such as towns and villages. Definite legislation is accordingly needed for effectuating correlated action throughout the metropolitan district, both in planning the transportation system as a whole and in detail, and in fixing priorities for the improvement program. It is necessary, therefore, to distinguish between such counties, and to recognize that to speak merely of the size of an administrative unit may be inconsequential, if not misleading.

Since such special considerations must be taken into account, it seems obvious that no definite standard-size unit can be prescribed which will be a universal absolute for highway administration. The intensity of highway needs varies, as well as the degree to which a region has been developed and the type of its development. Large agricultural regions might prove nearer the optimum unit for highway administration than large areas of concentrated industrial development. Physical characteristics such as topography and climate are important factors for consideration as well as possible sources of highway funds and probable necessary amounts of expenditures.

The optimum size of highway units.—Certain characteristics of local government mentioned are susceptible to correction, such as lack of planning, budgeting, and other administrative matters. It is claimed by the opponents of centralization that county government may be revived by effecting reform along these lines. But many criticisms against the local highway unit as an administrative body are functions of physical characteristics which are not susceptible to "reform." No matter how efficient its system of accounting nor how expert its highway commission, local government may still be limited to uneconomical operations unless it is able to raise sufficient funds to pay the highway bill and unless the scope of construction and maintenance requirements will allow fullest utilization of equipment, a proper distribution of overhead, and the economical operation of a competent engineering organization.

The economist recognizes that a profitable industrial plant is limited in its physical equipment to an optimum unit of operation: That unwieldy production units cause economies of large-scale production to give way to dis-economies, and that particular circumstances may alter the optimum plant even in the case of similar products. On the other hand, horizontal combination of a number of optimum production units under centralized administration is entirely in keeping with economical operation. The so-called American trust is an example of such horizontal combines. In other words an industry may require technical decentralization and managerial centralization.

This principle of economics appears to be applicable to the provision of highway facilities, in which optimum highway operating units might be determined upon, and their management directed centrally. Such is the general plan adhered to in the division of State highway systems into engineering districts, and suggested in the relation existing between the Federal and State governments.

It does not appear unworkable that all rural roads in a State might be operated on a similar basis. Each State might contain several highway operating units varying as to optimum sizes in accordance with particular considerations. These districts might be a grouping of counties or other local jurisdictions into regional areas. In small States or States essentially agricultural the entire area might be determined the optimum, in which case consolidation of all roads in the State would be economically in order. Whatever the size and number of operating units, however, financial and planning administration might still be centered in the State.

DATA BEING OBTAINED TO ENABLE PLANNING OF FUTURE HIGHWAY PROGRAMS

The establishment of the State highway departments was recognition of the need of centralized administration in creating a primary system of roads, and in the spending of State vehicle taxes with wisdom and coordination for the best interests of the whole State. Local units of government on the other hand were left to administer their individual highway affairs, which were truly local affairs financed by local money. With the State-wide extension of motor transport, however, all roads within a State developed into a network which it was necessary to view as a whole. Recognition of the wider influence of secondary roads was granted in the form of allocations of State money to local units of government which were not established to be expending agencies for such funds. Accordingly, the principle came to be tolerated that there should be centralization of certain highways in the State, financed by State funds, and decentralization of certain other roads, also financed with State taxes, in a multiplicity of lesser governmental units. There is basic conflict between these two policies. On the one hand it is accepted that the highways constitute a closely-knit system; on the other hand uncorrelated policies of finance develop them as a patchwork.

The chief objections to State control of all highways are for the most part political rather than economic. That is, there is general recognition of the possibilities of economy and a coordination with control centered in the State highway department, but there is fear concerning the effect on local government which might result from eliminating local highway administration. Such action, it is asserted, would tend to discourage interest in other local governmental functions and eventually to bring about complete State centralization. This would be the first step, according to stock arguments, toward the destruction of self-government, individual initiative, and democracy.

The "fine-woven rhetorical expressions" advanced in behalf of local government, it is pointed out, must be tempered with the common-sense observation that highway transportation is not a function properly confined to imaginary and outmoded political boundaries. To claim that the preservation of democracy depends upon the maintenance of such a system has been construed by some as an argument for governmental waste and inefficiency; and to extol the small local unit as a "school for democracy" has been challenged on the grounds that accounting and engineering are so often omitted from its course of study. The statement has been made that if democracy can coexist with such philosophies of government there is little fear that it would perish from State financial administration of highways.

Factors supporting centralization trend.—A consideration of importance with regard to the future possibilities of centralized highway administration is the recently inaugurated Federal assistance for secondary road development. During the depression years secondary roads and urban streets were granted various emergency appropriations by the Federal Government for the prime purpose of furthering employment. In the present fiscal year, however, regular Federal-aid grants of $25,000,000 are available for secondary road improvement, to be matched by equal amounts of State funds. It is of significance that the State highway departments may employ the services of competent county highway organizations acting under direction of the State, in the preparation of plans, surveys, and specifications, and in the supervision of construction. Where laws limit the State highway department in the extent of mileage it can maintain, the State may draw up agreements with lesser governmental units which will attend to the maintenance of these secondary roads. No such agreement will be approved, however, if any road previously built with Federal funds and currently maintained by a county or lesser political unit is not being kept in satisfactory condition.

Centralization and planning.—A further development toward closer cooperation between State and county, and greater control by the State over local roads is the promising possibility of State-wide highway planning. Surveys now under way to provide the facts necessary for plans may be made the instrument for publicizing the inadequacies of small highway units, and for revealing to the taxpayer how much of his money supports obsolete governmental machinery instead of better roads. It is also hoped that State legislation may follow the findings of such surveys when questions of highway administrative reform arise.

Some of the immediate purposes of the State-wide planning surveys are included in the following:

1. To define the mileage of roads within each State to be supported by public funds.
2. To determine the use made of the parts of this system, hence the sources of necessary taxes and their proper distribution.
3. To determine future construction requirements for extensions, improvements, and replacements.
4. To determine the priority of such construction projects.
5. To estimate necessary maintenance operations.
6. To estimate future highway income and to budget this sum according to estimated future financial requirements.

These several purposes emphasize the need for control by a central agency to supersede uncoordinated plans which result from the operation of a large number of highway jurisdictions acting independently. In order that planning may be effective throughout the State there must be an administrative control with greater power than any of the separate minor units. Planning which is "State wide" cannot be attained by a number of individual plans within the State, but only by a central plan which applies to an integrated system.

In review of the foregoing status and trends in State vehicle tax distribution for highways and in highway administrative procedure, a summary of the data is presented, followed by a list of conclusions and recommendations suggested by them.

1. Approximately one-fourth of all State motor-vehicle taxes were distributed for local road and street purposes in 1936.

2. The share of State funds allocated to local roads and streets has increased only 3.1 percent in the last 10 years, while the actual money so distributed shows a 115 percent dollar increase during the same period.

3. The State highway share of motor-vehicle taxes has decreased more than 17 percent in 10 years, while the dollar allotment has increased 42 percent.

4. State funds are distributed to local units of government in widely varying amounts and without regard to traffic generated, five States making no allocations and one distributing more than 24 million dollars.

5. Methods of distribution among each separate local unit are generally untenable, being made in equal amounts or on the basis of area, population, road mileage, assessed valuation, vehicle registrations, tax collections, or a combination of two or three of these.

6. In most cases the States retain no control, or merely nominal control, over the spending of vehicle taxes used on local roads and streets.

7. Four States have consolidated all rural roads in the State highway departments, while 26 States have State and county organizations, 6 have State and township units, and 12 have three systems: State, county, and township.

8. In the past 6 years 21 States have shifted 171,932 miles of local roads to State control, constituting a 64.3 percent increase in State mileage during that period.

9. More States were involved in local road consolidations in 1936 than in any previous year.

10. In the past 7 years 4 States have eliminated all township road units.

11. The highway consolidation movement has shifted the highway tax from local to State government and from property to motor vehicles.

RECOMMENDATIONS AND CONCLUSIONS

1. Allocation of State vehicle taxes to local roads and streets should be made with reference to both volume and intensity of traffic generated, but with consideration for the priority of primary road requirements so that transportation facilities for the integrated system may be adequate and at lowest total cost.

2. The State should maintain adequate control over all projects on which State money is used.

3. Arbitrary political boundaries have no relation to functions of highway transport.

4. A highway operating unit may be limited in its ability to function economically by reason of certain characteristics inherent in small-scale operations.

5. A highway administrative area is not necessarily limited to the optimum unit determined upon for construction and maintenance operations, and should embrace sufficient area to permit quantity purchasing, specialized personnel, and a coordinated highway program.

6. With the transfer of local roads to State control, benefits to land remain a legitimate highway service which should be recognized by property contributions to the highway fund.

7. It is important that the State should provide first for all primary road obligations before assuming added burdens in connection with local roads.

8. Federal aid for secondary roads is recognition of the fact that such parts of the highway system are of more than local service. This new Federal policy promises to create closer cooperation between States and local units.

9. State-wide planning surveys constitute the first wholesale attempt to bring before the public and legislative bodies facts concerning the need for sane financial and administrative policies.

10. State-wide plans cannot be successful without a central planning authority.

11. The failure of any State to provide a major system of highways not only adequate but attractive to the rapidly growing tourist and recreational traffic results in large losses of potential income to the public from the user taxes and to private business relying upon the highway travel.

The failure to establish and to follow sound principles of financial administration is a serious cause of lack of progress toward adequate major highways where this condition exists.

12. The waste of highway funds by duplicate local units and the uneconomical operations they necessitate brands financial administration the least progressive field of highway transportation.

SUMMARY

Last year approximately a quarter of a billion dollars, or one-fourth of total State motor-vehicle collections, was distributed for local road and street purposes.

Study of the past 10-year trend in this allocation of State funds reveals that the 1936 allotment was more than double the amount distributed to local jurisdictions in 1927. The percentage of total collections so distributed, however, has increased during that period by only 3 percent.

The amount of State funds spent on State roads has also increased in this 10-year period, but the increase has been only 42 percent as compared with the 115 percent increase in local road apportionments. Moreover, there has been an actual decrease of 17 percent in the share of total State taxes so used. This discrepancy appears to be a result of a wholesale use of funds for other-than-highway purposes.

The amounts of user taxes going to local units of government vary widely from State to State, as do the methods upon which such distribution is based. In 1936 five States made no allocations to local roads, while one State distributed two-thirds of all motor-vehicle receipts. Distribution among the local units was found to be based on a variety of criteria, including population, area, vehicle registrations, valuation, tax collections, road mileage, and combinations of these factors. In some States these funds are distributed equally among the local governments.

It has been found that these methods of local road allocations often fail to reflect properly the needs of the highway system as a whole. For an economic distribution of funds requires that money be spent according to the needs of traffic, expressed in terms of the lowest possible total cost of transportation, which includes not only road costs but vehicle operating costs. In other words it is not merely total traffic which must be considered, but the concentration of this traffic: The intensive as well as the extensive use made of the highway system. Moreover, since all needed improvements cannot be made simultaneously, funds must be spent according to a priority which will permit the largest reduction of total transportation costs to be made first. In determining this priority it should be remembered that because of the integration of traffic on several road systems, it is advisable to improve the primary system first, since it carries the largest amount of concentrated traffic and its improvement brings about increased travel and increased receipts for the support of local roads.

It has been found that the spending of State funds by local governments is not always to best advantage because it is not properly controlled by the State. In addition, a complexity of lesser units of government discourages broad improvement programs, coordination, and long-range planning. Many local units do not comprise sufficient taxable wealth and highway activities to qualify them as logical highway administrative agencies.

In the search for the proper scope for highway activities it is concluded that there may be a distinction between the highway administrative unit and the economic operating unit: That the former may comprise several of the latter. This principle is recognized to some extent in the relation between the Bureau of Public Roads and the State highway departments, as well as in the division of a State into State highway districts.

Operating units which do not have sufficient taxable wealth and traffic may require consolidation before they are able to perform their functions economically. Among other things there must be sufficient road work to allow efficient utilization of equipment, and sufficient appropriations to permit a competent engineering force.

Two types of counties are recognized: Rural and urban. Rural county highway units may comprise large areas for economic highway operations; while the urban county, because of its wealth, population, and traffic, may properly be confined to a small area. Because the urban county is usually part of a larger metropolitan area containing other counties, as well as towns and villages, immediate legislation is needed for effectuating correlated action, both in planning the transportation system of the region as a whole and in detail, and in fixing priorities for improvement programs.

Correction of the weaknesses of highway administrative finance, when left to the discretion of a large number of local governments, has been attempted by consolidation of road units, particularly by the transfer of local roads to State control. In the past 6 years 21 States have taken over 172,000 miles of local roads, constituting a 64-percent increase in State mileage during that period. Four States have eliminated all locally administered rural highways. As regards the other administrative set-ups, 26 States have State and county organizations, 6 have State and township units, and 12 have three systems: State, county, and township.

Transfers of local roads to State control have brought about a shift of the road burden from land to motor vehicles and from local government to the State.

Although the road-consolidation movement was precipitated by the recent economic depression, as a means of relieving property of the road-tax burden, it appears that the inherent failings of incompetent local governments have been underlying causes of the movement. For in 1936, 10 States effected local road transfers to their State highway departments, a larger number than in any previous year.

It is felt that the policy of Federal-aid appropriations for secondary roads, as well as the trend toward highway planning, will in many cases accentuate the movement for State administration of rural roads.

APPENDIX

TABLE A.—*Legal provisions regulating the use of State motor-vehicle funds for local roads and streets* [1]

GASOLINE TAXES

State	Tax rate	Distribution to local roads and city streets
	Cents	
Alabama	6	3 cents to counties, distributed equally.
Arizona	5	3/10 to counties, according to gasoline sales in each
Arkansas	6.5	7 7 percent to counties, on basis of population, registration, and area.
California	3	1/2 to counties; $5,000 for each county and county-city, 4 times per year; balance distributed according to registrations.
Colorado	4	27 percent to counties, 3 percent for extensions of State system in cities, towns, and counties; on basis of State mileage in counties.
Connecticut	3	
Delaware	4	
Florida	7	3 cents to counties, distributed among them by particular statutes.
Georgia	6	1 cent to counties on basis of State-aid mileage in each.
Idaho	5	
Illinois	5	1/3 to counties, 1/3 to municipalities, on basis of vehicles registered.
Indiana	4	40 percent to counties, 10 percent to cities; according to population.
Iowa	3	4/5 to counties, by area.
Kansas	3	
Kentucky	5	
Louisiana	5	
Maine	4	To general highway fund, with registration fees, from which $150,000 goes to town roads, $700,000 to third class roads, on mileage basis, and $1,000,000 to State-aid roads according to town valuation.
Maryland	4	1.05 cents to counties, by mileage of county roads; 1.15 cents to Baltimore city.
Massachusetts	3	
Michigan	3	To State highway fund, with registration fees, from which $6,000,000 goes to counties, 7/8 in proportion to fees collected, 1/8 equally.
Minnesota	3	1/3 to counties, based on mileage and traffic needs.
Mississippi	6	2½ cents to counties, on basis of population, registrations, and area.
Missouri	2	
Montana	5	
Nebraska	4	3/8 to counties.
Nevada	4	
New Hampshire	4	Small amount to some local roads (less than 9 percent of total in 1936).
New Jersey	3	$5,000,000 to city streets.
New Mexico	5	
New York	3	5 percent to New York City; 20 percent to counties, by mileage.
North Carolina	6	
North Dakota	3	1/3 to counties on basis of registration fees collected.
Ohio	4	3 cents, minus about $285,000, to counties, villages, and townships on basis of vehicles registered.
Oklahoma	4	1/4 to counties, according to population and area.
Oregon	5	
Pennsylvania	4	1/2 cent to counties, based on gas-tax returns during preceding 3 years.
Rhode Island	2	
South Carolina	6	1 cent to counties, based on registrations.
South Dakota	4	
Tennessee	7	To counties: 1 cent equally, ½ cent by population, and ½ cent by area.
Texas	4	
Utah	4	
Vermont	4	$500,000 to local roads, by mileage
Virginia	5	$239,000 in 1936 for the 3 counties not under State control.
Washington	5	3 cents to counties and cities, according to gas sales.
West Virginia	4	
Wisconsin	4	
Wyoming	4	25 percent to counties; based 30 percent on area, 30 percent on rural population, and 40 percent on assessed valuation.

REGISTRATION FEES

State	Tax rate	Distribution to local roads and city streets
Alabama		20 percent to incorporated municipality or county where owner resides.
Arizona		50 cents of original fee retained by county.
Arkansas		
California		Approximately 30 percent to counties in proportion to registrations.
Colorado		50 percent to counties in proportion to collections.
Connecticut		
Delaware		
Florida		
Georgia		

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[1] Data incomplete.

TABLE A.—*Legal provisions regulating the use of State motor-vehicle funds for local roads and streets*—Continued

REGISTRATION FEES—Continued

State	Tax rate	Distribution to local roads and city streets
Idaho	------	90 percent retained by counties.
Illinois	------	
Indiana	------	¼ to counties and cities; counties, ⅞ on mileage, ⅛ on population; cities, on basis of population.
Iowa	------	
Kansas	------	10 cents of each registration to county.
Kentucky	------	
Louisiana	------	
Maine	------	
Maryland	------	After debt service and operating expenses of motor-vehicle department, traffic court, etc., 30 percent to Baltimore.
Massachusetts	------	
Michigan	------	See gas-tax data.
Minnesota	------	
Mississippi	------	All to counties where collected.
Missouri	------	
Montana	------	All to counties where collected.
Nebraska	------	5 cents retained by counties for each original registration.
Nevada	------	
New Hampshire	------	Small sum for State-aid ($272,000 in 1936).
New Jersey	------	Carrier taxes to municipalities.
New Mexico	------	15 percent to counties in proportion to registrations.
New York	------	25 percent to counties.
North Carolina	------	
North Dakota	------	
Ohio	------	47 percent to counties where car registered.
Oklahoma	------	9 percent to cities, 51 percent to counties.
Oregon	------	
Pennsylvania	------	
Rhode Island	------	
South Carolina	------	
South Dakota	------	76½ percent to counties where collected.
Tennessee	------	
Texas	------	100 percent to county where collected, up to $50,000; 50 percent up to $175,000.
Utah	------	
Vermont	------	
Virginia	------	
Washington	------	
West Virginia	------	
Wisconsin	------	20 percent retained by town, village, and city; also $3,000,000 to counties for State-aid roads, 40 percent on basis of registrations and 60 percent by mileage.
Wyoming	------	County registration fees retained.

TABLE B.—*Disposition of State motor-vehicle receipts to State highways and local roads and streets, 1927–36*

Year	Total funds distributed	For State highway purposes	Percent	For local roads and streets	Percent	Total fund to State highways, local roads, and streets	Percent
REGISTRATION FEES							
1927	$301, 061, 132	$220, 645, 359	73. 3	$61, 543, 245	20. 4	$282, 188, 604	93. 7
1928	322, 630, 025	235, 142, 906	72. 9	66, 569, 311	20. 6	301, 712, 217	93. 5
1929	347, 843, 543	250, 704, 624	72. 1	73, 226, 339	21. 1	323, 930, 963	93. 2
1930	355, 704, 860	253, 013, 603	71. 1	74, 639, 463	21. 0	327, 653, 066	92. 1
1931	344, 337, 654	234, 593, 379	68. 1	79, 388, 101	23. 1	313, 981, 480	91. 2
1932	324, 273, 510	188, 539, 140	58. 1	83, 298, 207	25. 7	271, 837, 347	83. 8
1933	301, 315, 447	157, 754, 844	52. 4	75, 943, 682	25. 2	233, 698, 526	77. 6
1934	318, 576, 965	175, 382, 722	55. 1	84, 356, 966	26. 5	259, 739, 688	81. 6
1935	324, 855, 135	173, 477, 594	53. 4	87, 587, 250	27. 0	261, 064, 844	80. 4
1936	374, 921, 000	194, 491, 000	51. 9	98, 241, 000	26. 2	292, 732, 000	78. 1
Total	3, 315, 519, 271	2, 083, 745, 171	62. 8	784, 793, 564	23. 7	2, 868, 538, 735	86. 5
GASOLINE TAXES							
1927	$258, 966, 851	$188, 951, 526	73. 0	$61, 633, 115	23. 8	$250, 584, 641	96. 8
1928	305, 233, 842	225, 315, 715	73. 8	68, 562, 491	22. 5	293, 878, 206	96. 3
1929	431, 636, 454	318, 087, 598	73. 7	101, 961, 887	23. 6	420, 049, 485	97. 3
1930	494, 683, 410	359, 797, 465	72. 7	118, 247, 702	23. 9	478, 045, 167	96. 6
1931	537, 589, 717	381, 711, 610	71. 0	134, 318, 053	25. 0	516, 029, 663	96. 0
1932	514, 138, 900	336, 144, 197	65. 4	127, 220, 400	24. 7	463, 364, 597	90. 1
1933	519, 403, 450	314, 432, 266	60. 5	153, 777, 094	29. 6	468, 209, 360	90. 1
1934	565, 139, 596	333, 196, 930	59. 0	138, 338, 782	24. 5	471, 535, 712	83. 5
1935	615, 580, 975	348, 651, 966	56. 6	150, 546, 567	24. 5	499, 198, 533	81. 1
1936	683, 074, 000	389, 125, 000	57. 0	167, 255, 000	24. 5	556, 380, 000	81. 5
Total	4, 925, 447, 195	3, 195, 414, 273	64. 9	1, 221, 861, 091	24. 8	4, 417, 275, 364	89. 7
TOTAL MOTOR-VEHICLE TAXES							
1927	$560, 027, 983	$409, 596, 885	73. 1	$123, 176, 360	22. 0	$532, 773, 245	95. 1
1928	627, 863, 867	460, 458, 621	73. 3	135, 131, 802	21. 5	595, 590, 423	94. 8
1929	779, 479, 997	568, 792, 222	73. 0	175, 188, 226	22. 5	743, 980, 448	95. 5
1930	850, 388, 270	612, 811, 068	72. 1	192, 887, 165	22. 7	805, 698, 233	94. 8
1931	881, 927, 371	616, 304, 989	69. 9	213, 706, 154	24. 2	830, 011, 143	94. 1
1932	838, 412, 410	524, 683, 337	62. 6	210, 518, 607	25. 1	735, 201, 944	87. 7
1933	820, 718, 897	472, 187, 110	57. 5	229, 720, 776	28. 0	701, 907, 886	85. 5
1934	883, 716, 561	508, 579, 652	57. 5	222, 695, 748	25. 2	731, 275, 400	82. 7
1935	940, 436, 110	522, 129, 560	55. 5	238, 133, 817	25. 3	760, 263, 377	80. 8
1936	1, 057, 995, 000	583, 616, 000	55. 2	265, 496, 000	25. 1	849, 112, 000	80. 3
Total	8, 240, 966. 466	5, 279, 159, 444	64. 1	2, 006, 654, 655	24. 3	7, 285, 814, 099	88 4

TABLE C.—*Road consolidations*

Year	State	Local road mileage transferred to State	Year	State	Local road mileage transferred to State
1931	North Carolina	46, 826	1935	Delaware	2, 602
	Pennsylvania	20, 167		Nebraska	1, 391
	Louisiana	6, 658		Missouri	834
				Nevada	796
	Total	73, 651			
				Total	5, 623
1932	Virginia	37, 028			
			1936	Arizona	428
1933	West Virginia	29, 098		Georgia	648
	Oregon	2, 646		Kentucky	340
	California	6, 600		New Mexico	2, 021
				Ohio	2, 391
	Total	37, 744		Oklahoma	606
				South Carolina	419
1934	Minnesota	4, 356		Texas	579
	Missouri	937		Missouri	914
	Georgia	367		Pennsylvania	2, 350
	Indiana	871			
	Kentucky	659		Total	10, 696
	Total	7, 190	1927–36	Total transfers	171, 932

SNOW REMOVAL AND ICE TREATMENT ON RURAL HIGHWAYS

Reported by H. A. RADZIKOWSKI, Associate Highway Engineer, Division of Construction, Bureau of Public Roads

Modern Highways Are Designed so as to Minimize the Amount of Snow-Removal Work Needed to Keep Them Open to Traffic.

IN ORDER THAT the large volumes of traffic using their main highways can be served with facility and safety during the winter, all of the northern and western States annually spend substantial sums for snow removal and ice treatment. Operations during the winter 1936–37, the latest for which data are available, indicate the importance of this duty of the State highway-maintenance forces.

Data compiled from reports by 36 States show that they spent $17,099,626 to free 217,243 miles of main highways from ice and snow during the winter 1936–37. These data, given by States in table 1, show that the major equipment used in this work was as follows:

Equipment:	*Number*
Trucks	13, 634
Tractors	1, 316
Graders	2, 505
Plows for trucks:	
Displacement type	12, 784
Rotor type	209
Plows for tractors:	
Displacement type	1, 029
Rotor type	107
Power shovels	41

In addition to actual removal of snow, the States erected a total of 12,040 miles of snow fence to prevent snow from drifting onto the highways. In each of 7 States the mileage of roads cleared of snow exceeded 10,000 miles; in each of 5 States the cost of snow removal and ice treatment exceeded $1,000,000.

Average snowfall figures for each State, reported by the United States Weather Bureau, are shown in table 1. These data are average figures for the entire State, compiled from records at stations located in various parts of the State. The average snowfall during the 1936–37 season was less than that during the previous winter in 25 States and greater in 10 States. In 8 States in the snow area the average snowfall was less than 20 inches during 1936–37, and in 10 States it exceeded 60 inches.

Average snowfall figures, however, cannot be accepted as a true index of the extent of the snow removal problem. Snowfall in mountainous regions may greatly exceed the average reported for the State. In one State the snowfall was reported as 469 inches on a mountain pass kept open during the winter season. High winds may blow snow over cleared roads, making additional removal work necessary. A small amount of precipitation, in the form of sleet, may necessitate considerable ice-treatment work.

Although the primary purpose of snow removal is to enable traffic to move with facility and safety, prompt

TABLE 1.—*Snow-removal and ice-treatment data for the winter of 1936–37*

State	Snow removal and ice treatment supervised by	Comparative average seasonal snowfall from records in different sections of State [1]		Snow-removal equipment [2]							Mileage of snow fence	Mileage of roads cleared of snow	Total cost of snow removal and ice treatment	Remarks
				Plows for trucks		Plows for tractors		Trucks	Tractors	Graders				
		1935–36 season	1936–37 season	Displacement type	Rotor type	Displacement type	Rotor type							
North Atlantic States		*Inches*	*Inches*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Number*	*Miles*	*Miles*	*Dollars*	
Connecticut	State	45.5	21.3	343	---	---	---	343	---	---	29	2,700	302,900	
Maine	do	99.3	63.0	286	1	44	1	287	45	---	184	2,433	372,880	
Massachusetts	do	45.7	22.0	819	---	23	---	851	23	---	54	1,762	588,561	
New Hampshire	do	91.8	71.2	234	1	3	---	242	3	---	148	2,556	561,094	Chlorides, $36,656; plowing, $144,346; sanding $347,264; and snow fence, $32,828.
New Jersey	do	32.0	12.2	450	6	3	4	156	7	102	91	1,572	220,398	2 snow loaders also used.
New York	State and counties	69.5	59.5	1,189	19	240	17	1,552	224	79	1,167	12,077	1,642,393	$808,488 expended by counties for snow removal; $833,905 expended by State for sanding; 56 of 57 counties on State highway system reported.
Pennsylvania	State	56.4	29.0	1,232	18	83	24	1,250	107	239	1,906	16,729	2,707,555	
Rhode Island	do	24.7	10.2	110	---	---	10	110	10	---	15	762	58,730	$12,882 for snow removal; $45,848 for ice sanding.
Vermont	do	68.9	63.3	120	---	5	---	120	5	3	112	1,759	327,585	
South Atlantic States														
Delaware	do	34.1	8.9	130	---	3	1	146	4	5	23	2,500	14,369	
Maryland	do	43.2	23.8	249	11	1	3	437	74	15	130	3,955	99,131	
Virginia	do	33.4	14.1	508	2	74	---	510	71	14	8	9,225	31,327	Data refer to primary State system.
West Virginia	do	66.5	27.2	229	---	14	---	353	16	34	37	4,565	71,808	Some evergreen trees planted along the roadside for snowdrift control.
North Central States														
Illinois	do	28.2	14.0	827	4	---		827	---	---	250	12,638	511,740	
Indiana	do	25.7	18.4	504	2	1		506	4	---	11	8,848	156,253	
Iowa	do	50.9	31.1	663	13	7	3	620	10	280	1,481	8,902	1,107,863	
Kansas	do	10.8	16.2	167	---	39	---	167	39	140	480	7,631	125,316	
Michigan	do	62.7	38.0	385	6	8	17	445	25	---	714	9,226	869,044	Some evergreen trees were planted along the roadside for snowdrift control.
Minnesota	do	57.3	69.0	432	23	22	10	364	5	---	1,161	11,195	1,603,323	
Missouri	do	15.1	14.2	270	2	1	---	500	1	50	241	13,750	310,367	
Nebraska	do	35.7	26.6	135	1	16	---	139	50	337	737	8,160	184,087	
North Dakota	do	39.0	36.3	25	7	1	---	25	1	25	575	5,000	242,848	
Ohio	do	11.0	21.1	669	6	10	---	675	10	35	106	14,000	575,000	
South Dakota	do	13.9	16.8	47	---	35	1	47	39	140	281	5,687	610,000	
Wisconsin	do	61.0	43.8	923	8	228	---	967	270	406	1,169	10,023	1,118,495	About 8 miles of hedge planted for snowdrift control; 21 power shovels also used.
Western States:														
Arizona	do	6.6	20.1	21	2	1	5	31	11	30	6	1,000	44,618	
California	do	(3)	(3)	130	17	21	1	138	25	50	17	5,263	604,768	Ice-sanding operations covered approximately 1,500 miles of the State highway system.
Colorado	do	68.2	72.1	224	9	55		305	132	209	68	3,593	152,863	5 power shovels also used.
Idaho	do	82.1	75.3	167	5	7	3	169	10	30	196	3,557	355,126	2 power shovels also used.
Montana	do	53.2	51.5	141	11	4	---	155	4	10	271	4,623	157,000	
Nevada	do	42.9	52.7	74	8	1	4	82	5	---	19	2,575	127,900	2 power shovels also used.
New Mexico	do	26.1	28.0	2	---	6	1	17	---	46	3	1,045	29,098	
Oregon	do	43.3	61.2	223	9	---	2	232	2	15	40	6,345	307,632	1 power shovel also used.
Utah	do	64.0	76.7	156	---	52	---	164	52	34	135	4,573	409,318	Do.
Washington	do	54.7	69.9	259	16	3	---	275	3	20	41	3,471	371,380	3 power shovels also used.
Wyoming	do	78.4	67.3	135	2	12	---	157	26	97	131	3,543	123,856	6 power shovels also used.
Total				12,784	209	1,029	107	13,634	1,316	2,505	12,040	217,213	17,099,626	

[1] Snowfall figures compiled from U. S. Weather Bureau records.

[2] The number of displacement plows, rotary plows, etc., listed includes equipment reported as under the control of various States and counties, but does not include equipment owned by numerous other counties for which no data have been submitted or by townships, municipalities, transportation companies, and diverse business agencies.

[3] Not available.

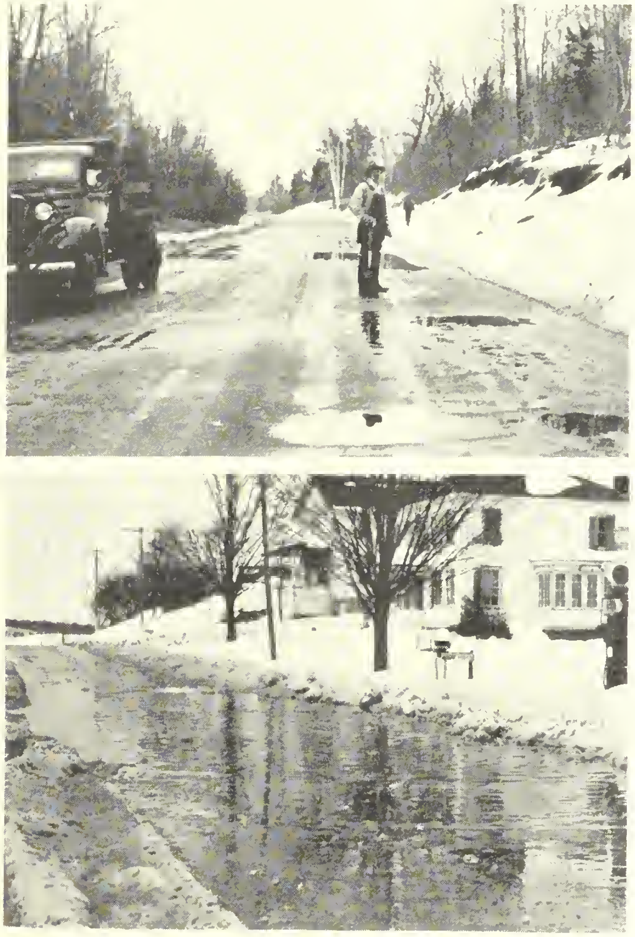
removal also helps to preserve the road surface and shoulders.

The snow should be pushed back from the shoulders to facilitate the flow of water from melting snow into drainage channels. Failure to remove snow from the shoulders, and allowing a thin cover of snow or ice to remain on the surface, results in erosion on low- and intermediate-type surfaces and loss of supporting value. The water from thawing snow and ice frequently runs along the edge of the pavement, softening the shoulders and allowing seepage under the pavement and into the subgrade. This excess water may serve to build up ice layers with resulting frost heave, and often cannot drain away through the ground because of an impervious layer of frozen soil below. The weakened road may quickly fail under traffic, especially at the edges of flexibile-type surfaces. It is now generally recognized that it is less expensive maintenance for snow-removal crews to preserve the road by draining the traveled way during each thaw than to repair winter-damaged surfaces and shoulders.

HIGHWAYS ARE DESIGNED TO PREVENT SNOW BLOCKADES

The problem of keeping the highways cleared of snow can be partly solved by using all practical means of preventing the snow from collecting on the roadway. Such preventive steps may include the erection of snow sheds, the planting of trees, erection of snow fences to control snowdrifts, and the design of the highway itself.

Highways in the northern and western States are located and designed with a view to minimizing the snow removal problem. The practice of locating the highway to take advantage of the prevailing wind is recognized. Road grades are raised above the adjacent ground level to provide wind-swept surfaces; slopes are flattened to reduce obstruction to wind-sweeping action; and ditches are widened to afford better drainage and greater snow-storage capacity. When funds permit, maintenance crews sometimes flatten and round through or side-hill cuts to create sections that induce air flow.



Improper Drainage During the Winter May Cause Damage to the Road and Create a Serious Hazard to Traffic.

Snow fences are used in areas where the topography is conducive to the formation of snowdrifts on highways. The purpose of these barriers is to cause the deposit of snow before reaching the road by retarding the velocity of snow-carrying winds. Location and placement of the fence, both temporary and permanent, are largely governed by the maintenance superintendents, who know where drifting has occurred in previous years. The portable picket-type of snow fence is most generally used (11,830 miles in 1936–37). This type of fence consists of wooden slats 4 feet high, 1½ inches wide, and ½ inch thick, woven together with galvanized wire and spaced 2 inches apart. The fence is secured to angle-bar posts on the windward side of the road a sufficient distance beyond the outside edge of the ditch to prevent the toe of the drift from covering the drainage channel. This type of fence is generally erected in the fall and removed in the spring. Trees and shrubs are sometimes planted as snow barriers where sufficient right-of-way is available. Brush and other obstacles that might cause snow to drift across the roadway are removed to allow free wind action.

Snow Sheds Are the Most Practical Means of Keeping Roads Open in Certain Areas Where Slides Occur.

Several different types of snowplows are used to remove snow from the highway. For light work graders and light blade-type displacement plows are adequate. V-type and side-wing displacement plows are suitable for moderately heavy work. For extremely heavy work rotor-type plows are generally used. As shown by table 1, the displacement-type plow pushed by trucks was the most widely used type during the 1936–37 season.

Light V-shaped and blade displacement plows are often used to open the traveled way for traffic, followed by rotary units which blow the snow clear of the roadway. Rotary plows and heavy tractor-propelled V-plows are used extensively in deep drifts on mountain passes. Use is also made of side-wing plows and slice bars for cutting down snow banks in deep drifts.

Motor Graders Are Often Used for Light Snow-Removal Work.

The clearing of mountain passes has been undertaken quite extensively by the various western States. The work usually requires the most powerful type of equipment to cope with the heavy snowfall, steep grades, high altitudes, and wind and slides. The auger blower rotary plow and the V-plows with rotary attachments for relieving the snow pressure on the face of the V are frequently used for this work. The rotors are generally driven by auxiliary power units carried on the rear end of the truck or by the tractor power unit

Displacement-Type Plows Pushed by Trucks Are Most Widely Used for Snow Removal.

which pushes the plow, depending on the type of motive force used. The rotors discharge the snow upward by centrifugal force. Deflecting chutes direct the discharge to either side, taking advantage of the direction of the wind. The snow is cast clear over the side banks, the distance it is thrown depending on the speed of the rotors and the force of the wind. In a few places where snow slides occur frequently snow sheds are the only practical means of keeping the roads open.

Efforts are made to prevent damage to guardrail and sign posts by snow equipment, by marking guardrails with high markers visible when the snow is deep, and by careful operation of equipment.

In 35 of the States reporting, the State highway departments controlled snow-removal work on State highways. In New York the counties performed snow-removal work, and the State did ice-treatment work on State highways. The States generally store snow-removal equipment at maintenance depots located at strategic points on the highway systems.

V-Type Snow Plow Suitable for Moderately Heavy Work

In the State of Washington, snow removal activities, especially in mountain passes, are controlled through the use of radio. By means of two-way, short-wave radio sets installed in snow-removal equipment, the central office dispatches units to snow blockades and directs the work by maintaining constant communication with the equipment. Supervisors' cars are also equipped with short-wave receiving sets, thus enabling the supervisors to be called to long distance telephone in cases of necessity for two-way communication. Shop clerks and operators of snow-removal equipment are encouraged to become licensed radio operators. These men carry on their regular duties while operating the radio equipment, and there is practically no expense for special operators to maintain the communication system.

SAND OR CINDERS ARE SPREAD OVER ICE-COVERED HIGHWAYS

Highways frequently become covered with ice during the winter months, rendering them impassable or extremely dangerous to traffic even at slow speeds. This can be remedied by spreading sand or cinders over the ice, thus restoring traction.

Ice may form as a result of a sleet storm, because of faulty snow removal, or because of faulty drainage. Until recently it was the practice to allow a covering of snow, ranging from a fraction of an inch to a few inches in thickness, to remain on the road surface. Compaction by traffic and alternate thawing and freezing soon transformed this snow into ice. With the increase of winter traffic and hazards from slippery roads there has been a change in practice on principal highways in many States; snow removal crews now scrape off the snow and slush as close as possible to the surface. Use is made of motor graders and light trucks with underbody blades mounted on springs for this work. This equipment is also used for surface and shoulder blading during other seasons of the year.

Icy Pavements Are Made Safer for Traffic by Spreading Sand or Cinders at Dangerous Places.

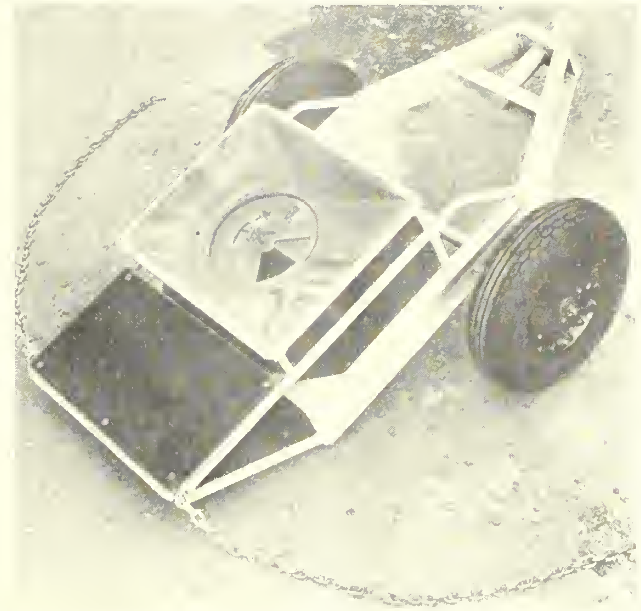
The formation of ice on pavements caused by water from thawing snowbanks is being eliminated through better snow-removal methods, by opening ditches and drains during thaws, and by a general study of the locations where ice forms so that the drainage facilities can be improved.

The hazards created by sleet storms or by rain precipitated at near-freezing temperature cannot be avoided. Temperature variations during the day, from temperatures permitting light rains and mists to freezing temperatures, permit a thin coat of ice to form within a very short time. The most effective remedies are: Provision of better traction by applying abrasives to the highway surface as soon as possible; protection with guard rail; the erection of suitable caution signs at approaches to critical sections; the elimination of excessive crown on pavements; and warning the traveling public by radio and other means as to the slippery condition of roads. Drivers should be informed of the things they can do to minimize the danger of skidding on icy pavements, such as carrying moderate air pressures in tires, using tires having deep treads, and driving at slow speeds so that brakes need be applied but seldom and then very slowly and cautiously.

The most commonly used abrasive is sand with an admixture of calcium chloride or sodium chloride on pavements other than portland-cement concrete. Cinders are preferred by some States, but are usually not available in sufficient quantities. The chemicals are used to assist in partially embedding the abrasives by temporarily lowering the freezing point.

The sand and chloride are generally mixed and stock piled at convenient points or sheltered in bins to prevent the weather from dissipating the salt and to keep the materials from becoming caked. Wherever possible the bins are placed in positions that will expedite handling of materials by gravity loading. Spreading is performed from trucks either by hand shoveling or mechanical spreaders, and directly from stock piles on short and isolated sections. Because of the cost, treatments are largely limited to steep grades, curves, grade crossings, intersections, and other dangerous places where accidents are most likely to occur.

The State of Oregon has developed a sand spreader for ice treatment. It is a trailer, power for spreading being supplied by traction of the trailer wheels through the trailer axle to a drive chain and gears which revolve two disks. The amount of materials spread and the width covered are regulated by manual adjustment of the disks. The trailer is attached to the rear axle, by which it is pulled, and also to the tail gate on the dump body of an end-dump truck. The trailer wheels do not come in contact with the pavement until the dump body is half raised. The abrasives fall through a chute in the tail gate into the feed hopper and onto the spreader disks. Recommended speed of operation is from 5 to 8 miles per hour, depending on the size of materials used. An automatic clutch arrangement near the wheel bearings permits backing of the trailer without operation of the drive mechanism. The machine was used experimentally during the winter of 1936–37, and reports indicate that about 50 machines of this type were constructed for use during the following winter.



Sand Spreader Developed and Used by the Oregon State Highway Department.

It has been reported that the oil film deposited on concrete pavement by passing vehicles seems to act as a protective coating against possible detrimental action of concentrated solutions of chlorides used in ice treatment. Several States are experimenting with linseed oil as a protective coating. In Wisconsin pavement is painted with two coats of the oil prior to the application of calcium or sodium chloride. Sufficient time has not elapsed to permit any definite conclusions, but it is reported that the State proposes to continue the experiments on a larger scale during the winter of 1937–38.

AUTOMOBILE PURCHASES BY FARM FAMILIES

Summary tabulations of a survey of 17,000 farm families in 64 counties in the United States show that 824 of each 1,000 families owned cars. The statistics, summarized in table 1, show that farm families buy nearly twice as many used cars as new cars. Usually these are relatively high-value used cars, although their cost averages only a little over one-third the cost of the new cars purchased—$263 for the average used car, $739 for the average new car.

The counties, surveyed in 1935–36, were representative of farm regions in all parts of the country. The data were collected in the Study of Consumer Purchases conducted by the Bureau of Home Economics of the United States Department of Agriculture as a Works Progress Administration project in cooperation with the Bureau of Labor Statistics of the United States Department of Labor, the National Resources Committee, and the Central Statistical Board.

Because a composite picture of the usual American family was the object of this family-living study, only families having both husband and wife were included.

Car ownership of all nonrelief, native white farm families interviewed ranged as high as 97 percent in California and in North Dakota and Kansas, where distances between cities are great. In Vermont the percentage was 73, still almost three-fourths. Among the white operators of the Southeastern States, more than 60 percent reported owning cars. For Negro sharecroppers, this percentage ran as low as 15. Car ownership the country over averaged a little over 82 percent.

Only in California did the white farm families studied purchase more new than used cars. In most of the other areas studied, twice as many used as new cars were bought. Farm families the country over paid an average of $263 for used cars, $739 for new cars. The purchase price of used cars ranged from $80 in Georgia and Mississippi to $330 in New Jersey; the price of new cars ranged from an average of $637 in North Carolina to $932 in California. Freight charges, of course, make a difference of $100 or more in car prices depending on the distance from the center of production. The price of used cars generally averaged well above $200. A few used cars costing $50 or less were bought, usually by low-income families.

For other areas the percentages of car ownership among the families interviewed were: Michigan and Wisconsin, 94 percent; Illinois and Iowa, 94 percent;

TABLE 1. *Number of nonrelief farm families [1] owning cars and number buying new and used cars, per 1,000 families; and average price of cars purchased, 1935–36*

Locality	Farm status	Race	Number of families in each 1,000 owning any car	Number of families in each 1,000 buying during the year: Any car [3]	New car	Used car	Average gross price [2]: New car	Used car
New England States:								
Vermont	Operators	White	734	140	41	99	$709	$186
Central States:								
New Jersey	do	do	881	139	64	76	761	330
Pennsylvania and Ohio	do	do	859	181	55	128	737	271
Michigan and Wisconsin	do	do	939	197	56	142	696	210
Illinois and Iowa	do	do	938	226	75	153	736	263
Mountain and Plains States:								
Kansas and North Dakota	do	do	965	182	55	127	717	247
Colorado, Montana, and South Dakota	do	do	861	215	107	110	807	303
Pacific Coast States:								
Central and southern California	do	do	971	203	107	96	932	304
Oregon and Washington	do	do	912	181	63	118	869	278
Oregon	Part-time operators	do	919	232	52	180	835	308
Southeastern States: [4]								
North and South Carolina	Operators	do	707	266	118	149	703	311
Do	Sharecroppers	do	448	175	27	148	669	212
North Carolina	Self-sufficing farm operators	do	204	53	10	43	637	223
North and South Carolina	Operators	Negro	425	139	21	118	638	197
Do	Sharecroppers	do	355	148	6	142	750	173
Georgia and Mississippi	Operators	White	622	178	73	106	736	240
Do	Sharecroppers	do	195	52	2	50	776	80
Do	Operators	Negro	246	51	0	51	0	181
Do	Sharecroppers	do	146	59	0	59	0	85
All regions			824	192	70	122	739	263

[1] Families that include husband and wife, both native-born.

[2] This average is based on the number of cars purchased during the year, for which the gross purchase price was reported.

[3] The figure in this column is the sum of the number of families buying new and those buying used cars, unless the same families purchased both types of cars.

[4] Because of the economic and social significance of the system of farm tenancy in the Southeast, these data have been tabulated separately for each type of tenure; hence no justifiable comparisons can be made between any one group in this region and white operators in other regions. Negro farm families were studied only in the Southeast.

Oregon and Washington, 91 percent. A special study of part-time farmers in counties adjacent to Portland, Oreg., showed that 92 percent were car owners. For New Jersey this percentage was 88; for Colorado, Montana, and South Dakota, 86; for Pennsylvania and Ohio, 86.

In the southeastern States, where the farm tenancy system was a factor, and where Negro families constituted a considerable proportion of the farm population studied, the percentages of car ownership for the families interviewed in North and South Carolina were: White operators, 71 percent; white sharecroppers, 45 percent; Negro operators, 42 percent; Negro sharecroppers, 36 percent. In Georgia and Mississippi the percentages were: White operators, 62 percent; white sharecroppers, 20 percent; Negro operators, 25 percent; and Negro sharecroppers, 15 percent.

A special study of the so-called self-sufficing farm area of North Carolina, where farmers raise more for home consumption than they do for market, showed 20 percent car ownership.

Car purchases during the year were reported by 14 percent of the Vermont families interviewed, 4 percent buying new cars and 10 percent buying used cars. Fourteen percent of the New Jersey families also were car buyers, 6 percent buying new cars and 8 percent buying used cars. Eighteen percent of the Pennsylvania and Ohio families purchased 5 percent new cars, 13 percent used cars. In Michigan and Wisconsin, 20 percent bought cars, 6 percent new cars and 14 percent used cars. In Illinois and Iowa, 23 percent bought cars, 8 percent new cars and 15 percent used cars. In Kansas and North Dakota, 18 percent bought cars, 5 percent new cars and 13 percent used cars. In Colorado, Montana, and South Dakota 22 percent bought cars, 11 percent new cars and 11 percent used cars. In California, 20 percent bought cars, 11 percent new cars and 9 percent used cars. In Oregon and Washington, 18 percent bought cars, 6 percent new cars and 12 percent used cars.

In the southeastern States, the new cars were almost all purchased by the white farm operators. In the four States studied in this region (North and South Carolina, Georgia, and Mississippi), less than 3 percent of the white sharecroppers, Negro operators, and Negro sharecroppers reported buying new cars. In Georgia and Mississippi, none of the Negro families reported any new-car purchases. Twenty-seven percent of the North and South Carolina white operators interviewed bought cars during the period studied, 12 percent bought new cars and 15 percent bought used cars. In Georgia and Mississippi 18 percent of the white operators' families purchased cars, 7 percent buying new cars and 11 percent buying used cars. Of all families interviewed the country over, 7 percent reported buying new cars and a little over 12 percent reported buying used cars.

FILM STRIPS ON HIGHWAY SUBJECTS AVAILABLE

Film strips relating to several different highway subjects have recently been made available by the Bureau of Public Roads of the United States Department of Agriculture. Lecture notes containing appropriate comments about each frame have also been prepared. Some of these notes are in the form of question-and-answer discussions by two readers.

"The Country Roadside Restored" is the title of one film strip of 55 frames. This series of pictures begins by showing the attractive roadsides in the old days of horse-drawn traffic. It continues by showing roadsides where beauty has been marred by construction scars, stumps, and unsightly debris. Later pictures show roadsides whose beauty has been retained by carefully planned construction operations or restored by the planting of trees, shrubs, and other plants, by the rounding and trimming of side slopes, and by the removal of unsightly stumps and debris.

Another series is entitled "The Cost of Poor Roads." The economy of good roads is forcefully illustrated in 53 frames, which show that lower car operation costs, superior highway service, greater community social values, and increased safety, all accrue when poor roads are improved.

Methods of stabilizing soil-road surfaces are shown in a strip of 55 frames entitled "Stabilized Soil Roads." Materials used in stabilizing road surfaces and the methods of preparing, mixing, and placing them, are illustrated in detail.

"Subsurface Exploration Using Electricity and Sound Waves" is the title of another series of 37 frames. It is important for highway engineers to have means of determining accurately the location and type of materials below the earth's surface without actually boring into the earth. Layers of rock, shale, clay, and earth offer different amounts of resistance to the passage of electric current and they also transmit sound waves at different speeds. The film shows clearly how, by making use of these phenomena, engineers can measure the depth below the ground surface to rock.

Investigations of highway materials, design, and methods of construction, are pictured in the 69-frame strip entitled "Highway Research Highlights." Roads must be designed to withstand the wear and tear caused by modern high-speed traffic and deterioration resulting from constant exposure to freezing, thawing, rain, snow, etc. Great progress has been made toward making highways stronger and more durable, and at the same time keeping costs as low as possible.

"Open Winter Roads," a 67-frame film strip, is devoted to the subject of snow removal on highways. In the days of horse-drawn traffic, winter snows offered no serious hindrance to travel. Sleighs and sleds were gotten out, and traffic moved on runners instead of wheels. Motor vehicles, however, cannot be operated safely or easily on snow or ice-covered roads. The pictures show the large snow plows, pushed by trucks or tractors, that are now used to remove snow from the highways. Sand or cinders are spread over curves, grades, intersections, and other dangerous places when they become coated with ice.

These pictures are on 35 millimeter, noninflammable film, suitable for projection in film-strip projectors. Suitable projectors may be rented in the large cities from one of the large telegraph companies. Projectors are available in most schools, and can often be rented from automobile dealers who use them in training their salesmen.

The film strips, together with lecture notes, may be borrowed from the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C. There is no charge other than the postage in returning the film (approximately 10 cents).

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF FEBRUARY 28, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,122,820	$551,410	64.3	$3,548,911	$1,775,426	156.0	$2,786,010	$1,391,580	122.4	$4,128,765
Arizona	1,689,169	1,221,427	81.6	1,559,947	1,064,457	80.2	563,888	371,176	30.6	462,763
Arkansas	2,852,611	2,847,004	183.8	1,281,294	1,277,611	85.9	23,505	23,177		2,251,127
California	6,772,548	3,738,976	150.0	7,163,159	3,765,555	116.8	1,959,290	1,046,167	40.6	358,556
Colorado	3,172,067	1,771,163	119.8	894,051	484,900	39.1	1,199,789	657,378	29.9	1,481,944
Connecticut	783,198	389,196	9.3	231,520	108,123	.7				1,513,589
Delaware	463,922	229,992	18.1	70,588	35,294	2.8	660,211	302,597	18.0	925,129
Florida	493,182	246,003	20.9	2,915,342	1,457,671	58.4	472,900	236,450	8.0	2,477,059
Georgia	2,110,923	1,020,343	122.5	5,810,489	2,904,836	263.3	1,761,388	880,694	82.0	3,822,664
Idaho	2,593,499	1,532,132	198.9	920,795	549,711	74.0	536,192	318,347	25.4	572,776
Illinois	10,190,017	5,023,351	298.1	5,994,861	2,846,273	147.3	5,816,950	2,908,400	85.9	178,523
Indiana	5,678,897	2,813,569	131.9	2,687,175	1,343,587	86.3	1,969,500	984,750	47.8	1,508,789
Iowa	6,475,119	2,959,914	211.8	4,214,635	1,887,856	126.2	2,471,860	1,151,450	68.5	94,596
Kansas	4,320,090	2,145,611	242.6	2,660,496	1,330,149	47.5	2,390,855	1,195,422	113.7	2,661,077
Kentucky	2,480,778	1,190,389	64.8	3,245,000	1,622,500	97.0	1,869,658	934,829	97.2	1,937,280
Louisiana	583,782	287,228	15.0	6,352,674	1,329,105	39.5	5,980,759	1,037,045	25.9	1,553,863
Maine	1,869,761	934,879	52.6	1,908,936	954,468	42.3	549,530	274,760	13.5	117,452
Maryland	930,330	465,145	13.4	1,814,594	905,534	28.9	460,360	230,180	.5	1,493,949
Massachusetts	4,403,913	2,201,955	20.3	1,880,495	940,247	3.3	636,468	318,234	5.0	1,627,896
Michigan	6,485,843	3,181,092	166.9	5,514,868	2,757,434	130.8	1,652,020	781,710	19.8	1,650
Minnesota	6,201,859	3,086,623	289.6	3,581,631	1,779,571	154.1	1,773,992	876,891	88.7	777,651
Mississippi	2,193,901	1,096,851	102.0	4,479,990	2,239,290	210.6	1,796,240	898,070	74.0	2,398,965
Missouri	8,206,746	4,055,492	446.8	4,719,135	2,271,015	126.4	3,072,726	1,221,271	74.5	1,292,071
Montana	3,824,816	2,144,049	277.7	1,846,412	1,038,231	86.4	571,734	321,594	33.8	1,991,430
Nebraska	2,464,943	1,230,188	255.9	4,393,751	2,184,059	398.4	3,565,182	1,152,278	179.1	1,577,131
Nevada	2,515,471	2,159,343	132.5	693,895	601,641	62.3	79,019	68,520	5.2	704,153
New Hampshire	360,696	177,963	6.5	498,834	246,525	9.5	838,179	394,706	9.9	615,625
New Jersey	2,055,958	950,684	20.3	2,136,030	1,067,695	14.6	519,680	258,955	5.3	1,209,232
New Mexico	2,916,628	1,783,013	229.0	2,324,936	1,517,997	116.6	25,662	15,531		244,516
New York	14,122,720	6,674,595	245.5	12,073,954	5,990,460	210.3	2,175,290	1,086,145	32.1	306,069
North Carolina	4,798,124	2,394,490	372.4	5,228,378	2,443,094	198.7	2,008,470	975,950	99.9	1,318,393
North Dakota	1,020,920	1,011,720	186.5	1,298,310	1,277,800	77.4	1,382,480	1,338,860	86.2	2,006,553
Ohio	3,658,876	1,758,941	51.0	9,308,609	4,617,756	100.5	1,733,320	866,660	18.7	4,859,087
Oklahoma	3,112,937	1,623,792	147.8	3,129,444	1,589,154	125.5	1,778,305	942,681	75.9	2,500,504
Oregon	3,973,517	2,354,927	130.9	1,535,252	897,609	81.0	519,952	317,270	15.0	718,809
Pennsylvania	11,946,751	5,944,779	168.9	7,146,528	3,553,824	99.8	2,705,525	1,351,725	50.6	2,024,731
Rhode Island	1,073,252	527,311	10.2	796,330	398,165	10.8	8,930	4,465		774,458
South Carolina	3,295,514	1,377,203	246.4	5,262,151	2,234,493	223.4	1,125,189	562,035	59.9	770,273
South Dakota	2,452,175	1,386,157	247.0	1,912,136	1,057,337	184.6	695,113	383,442	88.1	2,720,326
Tennessee	1,846,076	922,109	69.5	1,659,804	829,902	53.3	2,477,000	1,238,500	66.7	3,553,006
Texas	11,977,107	5,971,505	821.9	11,188,374	5,558,787	554.9	3,389,375	1,624,752	233.5	3,297,760
Utah	1,329,352	947,399	130.0	774,730	553,692	61.0	190,450	135,720	1.4	975,670
Vermont	1,052,720	505,707	29.6	1,679,730	739,095	43.1	6,730	3,168		61,552
Virginia	2,927,693	1,453,087	140.7	4,084,707	2,002,938	102.7	1,786,028	891,664	53.0	650,908
Washington	1,956,822	1,009,327	75.1	3,663,443	1,922,412	45.9	283,296	148,700	5.8	413,725
West Virginia	1,430,681	719,033	40.9	1,344,007	798,353	39.3	539,320	438,920	10.1	1,736,847
Wisconsin	8,140,567	3,918,966	269.6	4,701,047	2,159,177	107.5	635,299	307,800	11.1	884,384
Wyoming	2,465,437	1,412,048	256.0	1,355,569	822,160	151.8	174,090	107,550	16.6	197,137
District of Columbia										
Hawaii	640,163	311,939	11.5	431,329	212,667	6.4	640,320	318,175	12.9	985,427
Puerto Rico				947,719	471,880	20.0	127,820	63,910	.3	89,210
TOTALS	179,435,391	93,660,520	7,598.3	164,865,995	82,417,516	5,303.1	70,385,849	33,360,254	2,243.0	70,825,050

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF FEBRUARY 28, 1938

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,923,416	$ 3,874,665	136.9	$ 253,674	$ 247,774	7.8				$ 28,676
Arizona	2,569,841	3,153,137	2,501,556	193.7	38,548	38,548					29,737
Arkansas	3,352,061	3,163,049	3,133,003	351.6	165,042	164,942	8.6				54,115
California	7,747,928	7,954,378	7,579,423	263.9	138,156	138,156					30,349
Colorado	3,395,263	2,382,800	2,290,828	99.4	89,597	87,214	6.0	$ 8,200	$ 8,200		1,009,021
Connecticut	1,418,709	1,367,365	1,250,061	21.2	55,490	55,000	1.2	124,130	64,435	0.2	49,213
Delaware	900,310	871,470	843,920	66.4	10,234	10,234	.2	26,712	17,000	4.4	29,156
Florida	2,597,144	2,603,553	2,529,126	99.1	38,957	38,957					29,061
Georgia	4,988,967	1,962,958	1,912,758	112.3	2,358,364	2,089,984	109.8	497,320	497,320	28.3	488,905
Idaho	2,222,747	2,273,614	2,167,699	185.9	33,341	33,341					21,707
Illinois	8,694,009	8,135,128	7,867,135	465.1	685,010	685,010	23.2				141,364
Indiana	4,941,255	5,211,435	4,816,080	238.0	49,000	49,000					76,175
Iowa	4,991,664	5,241,284	4,885,935	528.3	105,902	104,865	.3				864
Kansas	4,994,975	4,695,661	4,642,082	370.6	251,823	249,650	21.6	46,162	46,162	.6	57,081
Kentucky	3,726,271	3,598,480	3,429,673	355.0	271,221	271,221	3.5				25,377
Louisiana	2,890,429	2,834,996	2,511,943	166.2	300,662	240,049	1.6	97,087	97,070	10.4	41,367
Maine	1,676,799	1,632,402	1,618,549	74.4	58,250	58,250	1.4				
Maryland	1,750,738	773,435	766,648	27.2	467,536	467,536	10.0	271,366	271,366	5.5	245,189
Massachusetts	3,262,885	2,219,079	2,218,903	18.2	896,550	458,710	.6	1,042,586	521,293	.4	63,979
Michigan	6,301,414	6,658,139	5,949,537	288.6	284,921	284,921	3.3	223,938	63,746	.3	3,210
Minnesota	5,277,145	6,383,720	5,170,077	896.0	93,670	85,850	5.8				21,218
Mississippi	3,457,552	3,152,229	3,147,534	220.8	224,146	223,106	14.9	10,800	10,800	.6	76,112
Missouri	6,012,652	5,294,408	5,211,301	777.0	790,784	736,082	.7	34,390	32,294		32,975
Montana	3,676,416	3,593,576	3,563,290	200.7	95,385	95,385	.1	8,462	8,462		9,279
Nebraska	3,870,739	3,432,726	3,321,652	362.3	446,353	446,353	8.3	70,270	70,270	1.8	32,464
Nevada	2,243,074	2,309,014	2,197,401	110.1	84,970	38,146	1.7				7,527
New Hampshire	945,225	875,636	843,253	37.7	121,092	101,301	5.7				671
New Jersey	3,129,805	1,598,383	1,577,527	31.1	1,512,979	1,512,979	4.4	34,468	34,468		4,831
New Mexico	2,871,397	2,811,309	2,805,816	213.7	43,071	43,071		14,681	12,196		10,314
New York	11,046,377	10,726,589	10,261,169	170.0	316,300	276,300	1.9	62,700	62,700	.2	446,209
North Carolina	4,720,173	4,643,501	4,574,722	285.4	120,761	120,761	5.4				24,690
North Dakota	2,867,245	2,488,483	2,440,577	377.9	107,799	107,799	1.2	290,601	290,601	37.5	28,268
Ohio	7,670,815	6,970,681	6,855,102	291.9	717,532	711,532	6.6				104,181
Oklahoma	4,580,670	4,565,720	4,321,366	400.8	239,470	239,470	7.6	8,800	5,000	.1	14,834
Oregon	3,038,642	3,136,986	2,975,094	164.6	45,580	45,580		11,846	11,846	1.0	6,122
Pennsylvania	9,347,797	7,872,088	7,302,979	264.3	2,110,819	1,854,586	21.1	36,737	36,737	.4	153,495
Rhode Island	989,208	1,113,140	989,208	18.8							
South Carolina	2,702,012	2,356,294	2,150,784	226.2	551,692	498,752	23.4	9,664	9,664	1.5	42,811
South Dakota	2,976,454	2,686,769	2,683,046	482.9	270,437	270,437	22.0				22,971
Tennessee	4,192,460	3,531,595	3,480,417	135.3	679,103	679,103	15.5	35,710	22,060	2.6	10,881
Texas	11,989,350	12,592,147	11,536,611	1,107.5	497,808	385,058	13.0	23,100	23,100	4.3	44,581
Utah	2,067,154	2,161,834	1,925,059	207.7	112,055	112,055	.1				30,040
Vermont	924,306	1,052,209	897,536	23.2	13,865	13,865					12,905
Virginia	3,652,667	3,475,512	3,287,636	939.9	131,731	128,434	13.5				236,597
Washington	3,026,161	3,316,821	2,940,420	164.2	65,824	65,824	.7				19,916
West Virginia	2,231,412	2,109,315	1,963,773	86.4	275,022	267,639	14.1				
Wisconsin	4,823,884	5,217,220	4,727,229	343.4	91,394	89,300	.3				7,355
Wyoming	2,219,155	2,173,387	2,165,009	152.4	33,287	33,287					20,359
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	946,320	862,482	17.4	9,533	8,907		62,530	54,644	.6	
TOTALS	195,000,000	184,243,391	173,917,090	12,780.4	16,354,740	14,964,324	387.1	3,052,260	2,271,434	100.7	3,847,152

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF FEBRUARY 28, 1938

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,546,325	$ 3,530,996	47	1	12	$ 503,819	$ 437,519	5			$ 53,400	$ 53,400	1			$ 12,702
Arizona	1,256,099	1,278,187	1,216,371	15		5	18,841	18,841									20,887
Arkansas	3,574,060	2,880,555	2,873,454	51	6	30	638,836	638,131	5		2	3,535	3,535			1	58,940
California	7,486,362	7,187,501	7,008,095	46	8		414,005	414,004	1			10,000	10,000	5			54,263
Colorado	2,631,567	2,287,197	2,218,984	27	1	21	411,634	411,634	3	2							949
Connecticut	1,712,684	479,557	479,463	3	1	1	1,157,071	1,135,710	7								97,511
Delaware	418,239	130,000	128,144	1			279,052	279,052	2								11,043
Florida	2,827,883	2,412,602	2,384,600	30	5	3	119,018	119,018	1			133,300	133,300			73	190,965
Georgia	4,895,949	809,234	807,128	17	7	16	1,514,089	1,514,089	24	7	3	674,070	674,070	10	1	33	1,900,662
Idaho	1,674,479	1,394,680	1,366,439	20	3	12	252,696	252,696	2		6	4,261	4,261			3	51,083
Illinois	10,307,184	8,785,913	8,659,208	65	8		1,500,134	1,493,730	9			101,000	101,000	2			53,246
Indiana	5,111,096	4,369,155	4,229,336	39	15		857,447	857,447	3		162						24,313
Iowa	5,600,679	4,568,177	4,474,833	98	9	8	1,092,875	1,089,000	8								36,846
Kansas	5,246,258	4,340,606	3,916,002	54		5	1,273,119	1,253,704	4	1		116,072	23,068	2		3	53,484
Kentucky	3,672,387	1,358,467	1,339,494	19	5		1,984,219	1,694,487	5	6		612,579	612,579	3	1		25,827
Louisiana	3,213,467	1,568,629	1,568,621	17	1		880,600	880,580	8	1		649,599	600,110	4	1		164,155
Maine	1,426,861	1,266,779	1,263,012	19	1	4	160,788	142,540	1	1							21,309
Maryland	2,061,751	665,089	660,481	5	3	18	881,174	831,174	5	2	11	283,163	282,277	1	1	1	287,819
Massachusetts	4,210,833	2,993,173	2,990,739	22	4		865,680	865,680	4			249,991	249,991	1			104,423
Michigan	6,765,197	6,864,894	6,560,765	44	8		101,477	101,477				167,000	75,730	1	1		27,225
Minnesota	5,395,441	4,824,965	4,793,348	83	13	50	590,070	584,020	3								18,073
Mississippi	3,241,475	2,326,914	2,321,760	51	5	11	445,900	445,900	6	2	2	217,000	217,000	2		10	256,815
Missouri	6,142,153	4,656,996	4,489,953	41	1		1,633,740	1,633,740	8		1	1,650	1,650			1	16,810
Montana	2,722,327	2,556,515	2,513,626	37	7		245,576	185,931	1	1							22,770
Nebraska	3,556,441	2,885,494	2,841,119	76	3	21	470,813	470,813	5		1	199,683	199,683	3		2	44,827
Nevada	887,260	877,030	845,455	8	3		13,308	13,308				3,630	3,630			5	24,867
New Hampshire	822,484	791,275	791,208	9	4	6	53,297	29,170	1			2,106	2,106			1	
New Jersey	3,983,826	3,107,456	3,094,275	20	6	1	772,334	772,334	4	3		44,730	44,730		1		72,487
New Mexico	1,725,286	1,700,885	1,693,233	19	1		25,879	25,879									6,173
New York	13,577,189	11,553,383	11,223,255	35	45		2,016,770	2,016,770	10	4		155,700	155,550		1		181,614
North Carolina	4,823,958	3,627,198	3,606,819	50	18		1,081,059	1,081,059	12		115						136,080
North Dakota	3,207,473	2,815,524	2,809,317	55	4		389,012	389,012		1							9,144
Ohio	8,439,897	2,476,674	2,355,458	19	6	8	5,664,564	5,196,418	30	2	5	670,000	655,000	10		1	233,021
Oklahoma	5,004,711	3,696,896	3,681,634	58	9	18	1,365,444	1,232,344	7	3	17	17,850	17,850			6	72,883
Oregon	2,334,204	2,291,480	2,249,772	16	6	2	74,215	74,215	1								10,217
Pennsylvania	11,483,613	8,357,476	7,754,595	69	18	9	3,568,378	3,291,343	17	5		150,473	150,473	5			287,202
Rhode Island	699,691	701,817	692,910	4	3												6,781
South Carolina	3,059,956	1,822,071	1,794,712	37	11	26	791,309	791,309	8	5	10	142,578	142,578	1		12	331,358
South Dakota	3,249,086	2,858,119	2,856,522	59	6	54	175,186	175,186	6			269,350	210,393	17			6,985
Tennessee	3,903,979	2,369,028	2,360,347	37	3	32	1,200,670	1,200,670	8	1	3	106,690	106,690	1		5	236,272
Texas	10,855,982	9,952,641	9,939,742	127	14	110	490,931	490,931	1	1	18	63,790	63,790			21	361,519
Utah	1,230,763	1,203,470	1,196,574	17	1	1	18,461	18,461									15,728
Vermont	729,857	751,916	706,638	10	7	20	10,900	10,900									12,319
Virginia	3,774,287	3,000,218	2,864,689	45	19	21	851,838	845,213	7	2	1	8,006	8,006			2	56,378
Washington	3,095,041	2,710,796	2,687,611	22	11	11	368,494	368,069	1	1							39,361
West Virginia	2,677,937	1,286,581	1,284,613	11	3	6	1,311,022	1,310,522	15			78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,499,995	4,462,992	37	6	7	395,129	394,926		1	11	91,598	24,151			16	140,614
Wyoming	1,360,841	1,213,957	1,203,739	13			111,212	111,212	1								45,890
District of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			179,710	169,698	2								
TOTALS	196,000,000	150,806,160	147,486,890	1710	319	549	39,221,795	37,759,866	251	52	358	5,281,659	4,905,456	71	7	197	5,847,788

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 2 ▽ APRIL 1938

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 2

April 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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RURAL AND URBAN CONTRIBUTIONS TO HIGHWAY TRAVEL AND EXPENDITURES

BY THE DIVISION OF CONTROL, BUREAU OF PUBLIC ROADS

Reported by ROBERT H. PADDOCK Associate Highway Engineer-Economist

THE steadily mounting emphasis on the function of highway transportation in the national life has directed increased attention to and investigation of the sources of funds with which the existing highway systems have been constructed, of the use and benefits derived from the highway systems, and of the probable amount and source of future funds necessary for the maintenance and improvement of the present system.

Since 1930, financial surveys, with particular reference to these phases of the highway problem, have been made in several States by the Bureau of Public Roads in cooperation with the State highway departments and the University of Wisconsin. In four of these States (Wisconsin, Minnesota, New York, and Colorado) special inquiries into the constituent elements of road use were made as a part of the financial analyses. These investigations enable analyses to be made that may point toward an adequate solution of the problem of the present and future financing of the entire highway system.

These studies are among the first that have attempted to obtain an evaluation of the benefits derived from taxes paid for the construction and maintenance of all roads and streets within a State. One of the first attempts to obtain data with which such an evaluation could be made appears to have been a study conducted in Wisconsin about 1916, under the direction of A. R. Hirst, State highway engineer, who reported in part as follows:

> A careful inquiry (through written question sheets) among automobile owners indicates that the average distance traveled by each automobile is at least 3,500 miles per year on roads outside the limits of incorporated cities and villages. If we estimate 140,000 pleasure cars in use in Wisconsin next year, which seems conservative, and each travels this number of miles, the motor travel on Wisconsin rural highways will be 490,000,000 miles. This does not take into consideration the travel of automobiles from other States. * * *
>
> It should be remembered that the total mileage traveled is being vastly increased each year. If we assume only 225,000 automobiles and trucks used for rural hauling in use in 1926 and that they will travel only 3,500 miles each, the total rural travel in 1926 will be 787,500,000 miles and the saving in that year to automobile owners alone, if a system of good roads was completed by that time, would be $7,875,000.[1]

While this early study indicates cognizance of the problem, it also indicates, by contrast with present conditions, the tremendous increase in importance that the problem has assumed in the succeeding 2 decades. Only 14 years after the above report was written the total vehicle-miles of travel on the rural highways of Wisconsin was approximately 10 times the amount estimated to have been traveled in 1916.

Although the main highways of nearly all States have been brought to a much higher degree of improvement in the intervening time there is an articulate current demand for more extensive highway development than is possible with the revenues now available. It is essential, therefore, to consider all factors that influence the need of highway facilities by different classes of highway users. These factors must include consideration of the ability to finance the desired system, and the equity of any taxation system established for that purpose. Investigations of the amounts of traffic carried by the several highway systems and the sources of that traffic are prerequisites to the determination of a sound financing policy.

HIGHWAY TAXES AND HIGHWAY USE BY URBAN AND RURAL RESIDENTS DETERMINED

The complete analyses of the highway financial problem accompanied by the road-use investigations made in Wisconsin in 1930, in Minnesota in 1933, and in Colorado and New York in 1934, form the basis of this comparative study.[2]

Data for the road-use survey were obtained in each State by a large number of personal interviews with representative motor-vehicle owners throughout the State, the sample being apportioned according to the distribution of vehicles in the respective rural and urban areas.

An estimate of the monetary importance of highway traffic within a State may be gained from table 1, which shows the estimated total vehicle-miles traveled annually in each of the States included in this analysis and an evaluation of the operating cost of this travel based on the conservative figure of 3 cents per mile.

TABLE 1.—*Estimated total travel within the State of residence by motor-vehicle owners, and estimated transportation cost incurred*

State	Year [1]	Travel in State by resident motor-vehicle operators	Estimated transportation cost at 3 cents per mile
		Vehicle-miles	
Colorado	1934	1,967,800,000	$59,034,000
Minnesota	1933	5,131,700,000	153,951,000
New York	1934	[2] 19,472,700,000	584,181,000
Wisconsin	1930	5,997,000,000	179,910,000

[1] All vehicle travel data for the 4 States given in this analysis will be for the respective years shown in this table.

[2] Residents of New York City accounted for 6,456,300,000 vehicle-miles or 33.2 percent of the total.

Annual vehicle operating expenditures of such magnitude make it desirable to answer the following questions:

1. By whom—that is, by residents of which governmental units—are these amounts spent?
2. For travel on what roads are these outlays made? (While present data make it difficult to evaluate exactly the relative earning power of the several highway systems, the proportional share of the travel carried by the several systems can be determined).
3. What expenditures are made for the highway system to provide for this travel?
4. Who pays for the construction and maintenance of the highway system, and what is the relation of such

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[1] Page 34, Third Biennial Report, Wisconsin Highway Commission, 1916.

[2] A summary of the Wisconsin survey was published in PUBLIC ROADS, vol. 14, no. 2, April 1933; the complete Minnesota report was published by the Minnesota Department of Highways and a summary appeared in PUBLIC ROADS, vol. 17, no. 1, March 1936. The complete New York report was published by the New York State Division of Highways and was summarized in PUBLIC ROADS, vol. 17, no. 9, November 1936. The Colorado report has not yet been printed.

expenditures to the use these individuals or residents of rural and urban areas make of the highways?

5. Who should bear the cost of present and future maintenance and extension of the system?

6. Who is best able to bear such costs?

Taxation for highway purposes has assumed a different aspect than most other forms of taxation because highway users are taxed in direct proportion to the extent of vehicle use. The amounts so collected have directly affected the extent of the traffic facilities provided. An investigation into the use of the facilities financed by these taxes seems warranted.

Inquiries to determine the extent of use of the various highways indicate that in each of the four States studied almost half of the total travel is performed on the primary highway systems, and that, except in New York, more than one-fifth of the total travel is performed on the secondary system and on the purely local rural roads—constructed and maintained either by the township or county. Tables 2 and 3 show the percentages of travel performed on each of the highway systems of these States on the basis of data obtained in road-use investigations.[3] In table 2 the amount of travel on urban streets that form part of the respective primary and secondary highway systems has been included with the total travel on those systems, while in table 3 the travel so performed has been allocated to the respective urban streets. In the latter table, only travel on the rural sections of the systems is included in the primary- and secondary-road travel.

TABLE 2.—*Percentage of total travel by motor-vehicle owners on the highway systems of their respective States*

State	Percentage of travel on—				Total
	Primary highways [1]	Secondary highways [1]	Local rural roads	Urban streets	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Colorado	48.9	13.4	8.9	28.8	100
Minnesota	44.4	19.8	6.1	29.7	100
New York	44.8	5.9	2.6	[2] 46.7	100
Wisconsin	52.7	13.6	7.9	25.8	100

[1] Data for the primary and secondary highways include travel on the urban extensions or connecting sections of the rural portions of the respective systems.

[2] 59.4 percent of the total local travel on New York urban streets was performed on streets in New York City. Thus, 27.8 percent of all travel on urban streets, excluding urban extensions of rural highway systems, was performed in New York City.

TABLE 3.—*Percentage of total travel by motor-vehicle owners on the rural and urban roads and streets of their respective States*

State	Percentage of travel on—				Total
	Primary rural roads [1]	Secondary rural roads [1]	Local rural roads	Urban streets	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Colorado	42.0	12.3	8.9	36.8	100
Minnesota	34.5	18.5	6.1	40.9	100
New York	30.6	4.1	2.6	[2] 62.7	100
Wisconsin	[3] 52.7	[3] 13.6	7.9	25.8	100

[1] Only travel on rural portions of the primary and secondary highway systems included. Travel on the urban streets serving as connecting links is shown under travel on urban streets except in Wisconsin.

[2] 49.4 percent of this travel, or 31 percent of all urban travel was on New York City streets.

Includes travel on city and village streets which served as connecting links for the rural systems.

Extensive systems of roads and streets were needed to serve the volumes of traffic shown in table 1. The motor-vehicle registrations of these four States and their respective street and highway mileages are shown in table 4. It shows that the streets and highways of New York accommodated the greatest number of vehicles per mile of highway. It would be expected, therefore, that expenditures per mile of highway would be greatest in that State.

In determining road-use relations highway users are divided into rural and urban groups, since highway taxation and highway use are distinctly different for these two groups. In this study, urban groups include all incorporated places (municipalities) within the respective States; the rural areas comprise the remainder of the State. Certain areas having urban characteristics but existing without benefit of incorporation are found in each of the States, but it is impossible to separate them for comparative fiscal purposes from the more inclusive rural governmental units containing them. The number and importance of such urbanized areas in the States studied are, however, relatively small. All comparisons made will be between rural and urban areas of each of the States, though certain travel characteristics in specific urban areas studied in the road-use survey will also be given.

TABLE 4.—*Population, motor-vehicle registration, and street and highway mileages*

State	Year [1]	Population (1930)	Motor-vehicle registration	Street and highway mileage	Persons per vehicle	Vehicles per mile of road
Colorado	1932	1, 035, 791	285, 860	52, 614	3. 6	5. 4
Minnesota	1932	2, 563, 953	704, 896	[2] 111, 475	3. 6	6. 3
New York	1932	[3] 12,588,066	[4] 2, 296, 063	[5] 100, 593	5. 5	22. 8
Wisconsin	1930	2, 939, 006	794, 404	89, 539	3. 7	8. 9

[1] All data on highway mileages, highway expenditures, motor-vehicle registrations, and all fiscal data, will be for the years given here for the respective States.

[2] City and village street mileage, except that on State or county connecting routes, was not available in this study.

[3] 6,930,446 persons in New York City, or 55.1 percent of the State total.

[4] 797,101 vehicles in New York City, or 34.7 percent of the State total.

[5] 5,271 miles in New York City, or 5.3 percent of the State total.

TREND TOWARD FINANCING PRIMARY HIGHWAYS FROM HIGHWAY-USER REVENUES OBSERVED

The data presented in table 4 suggest the relative magnitude of the highway problem in these four States. Later tabulations show the relative use of the available facilities and the source and amount of expenditures on the respective State systems.

While the percentages shown in tables 2 and 3 indicate the relative use of the several highway systems in each State, it is necessary for comparative purposes to determine the amounts and sources of expenditures on each of these systems. Table 5 shows the total amounts of expenditures on each of the highway systems of the four States in the year studied. Table 6 shows similar data but segregates expenditures on the rural portions of the several systems from expenditures on the urban extensions[4] in order to distinguish rural and urban expenditures according to location of the roads and streets. Tables 5 and 6 are similar in the classifications used to tables 2 and 3, respectively, and have been set up so that direct comparisons can be made.

The various networks of highways as now constituted in the several States are never exactly comparable as to function, demands on their facilities, or physical characteristics. This is especially true of the relation of the existing primary system to the entire highway system in the respective States. Table 7

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[3] The expenditures and travel on urban streets in this and succeeding tables refer to expenditures and travel on the streets of all incorporated places or municipalities within the respective States.

[4] Except for Wisconsin where such a separation was not possible.

TABLE 5.—*Amount and percentage of total road and street expenditures, by highway systems*

State	Expenditures made on—								Total	
	Primary system [1]		Secondary system [1]		Local rural roads		Urban streets			
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Colorado	$5,563,600	43.9	$2,065,200	16.3	$3,012,000	23.8	$2,035,500	16.0	$12,676,300	100
Minnesota	32,149,900	57.7	12,720,700	22.8	3,894,300	7.0	6,967,700	12.5	55,732,600	100
New York	68,094,300	31.6	38,284,400	17.8	22,531,100	10.4	[2] 86,673,400	40.2	215,583,200	100
Wisconsin	24,918,700	40.7	9,449,100	15.4	13,910,900	22.7	13,001,700	21.2	61,280,400	100

[1] Includes expenditures on urban extensions of the rural portions of the system (same classification of these extensions as in table 2).
[2] $68,587,500 or 31.8 percent of the total was expended on streets in New York City.

TABLE 6.—*Amount and percentage of total road and street expenditures in each State, by class of road and street* [1]

State	Expenditures made on—								Total	
	Primary rural roads [2]		Secondary rural roads [3]		Local rural roads [4]		Urban streets			
	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Colorado	$5,211,300	41.1	$1,975,500	15.6	$2,977,700	23.5	$2,511,800	19.8	$12,676,300	100
Minnesota	28,437,800	51.0	12,191,000	21.9	3,894,300	7.0	11,209,500	20.1	55,732,600	100
New York	52,489,200	24.3	37,565,600	17.4	22,531,100	10.5	[5] 102,997,300	47.8	215,583,200	100
Wisconsin	[6] 24,918,700	40.7	[6] 9,419,100	15.4	13,910,900	22.7	13,001,700	21.2	61,280,400	100

[1] Urban extensions of primary and secondary highways included with urban streets, as in table 3.
[2] State highways except in Colorado where only Federal-aid roads on the State system are included.
[3] County roads except in Colorado where they consist of other State highways not included in the primary system.
[4] Township roads except in Colorado where they are county roads. There are no township roads in Colorado.
[5] $73,229,300, or 34 percent of the total was expended on New York City streets.
[6] Includes expenditures made on city and village streets forming a part of the primary and secondary systems.

shows the percentage that the primary system is of the entire rural highway system of each State, and also shows the population per mile of primary road.

It will be seen from tables 2, 3, 5, and 6 that considerable disparity existed between the percentage of funds spent on the primary rural roads in the several States and the relative amount of travel on those roads. Only in Colorado was there close agreement between the relative portions of travel and expenditures on the primary roads. In the other three States considerable differences existed: In Minnesota the proportion of expenditures on the primary rural roads was 16.5 percent greater than the proportion of total travel carried on that system; in Wisconsin the percentage of travel exceeded the percentage of expenditures on the primary system by 12 percent. Similarly wide variations existed between travel and expenditures on the other road and street systems of each State.

TABLE 7.—*Ratios between mileage of primary highways and mileage of all rural highways, and population per mile of primary highway*

State	Percentage that primary highway mileage is of all rural highway mileage in the State	Population per mile of primary highway
Colorado	7.2	290
Minnesota	6.1	379
New York	14.9	1,015
Wisconsin	12.3	288

Before the initiation of highway-user taxation, and even until quite recently, streets and roads were very largely financed with general property taxes. Under such a system it was unavoidable that inequities in the tax burden should exist and that individuals or groups were taxed to support services from which they received little apparent benefit, or of which they made little immediate or direct use. The shift of the source of revenues for highway construction and maintenance from general property taxes to motor-vehicle and motor-fuel taxes has been considerably accelerated in recent years; but while highway-user taxes now finance most of the primary State highway work in these four States, a large part of other road and street expenditures is still financed with general property taxes.

Though it is frequently contended that the primary highways should be supported entirely out of highway-user taxes, in almost one-third of the 48 States, State revenues from motor-vehicle registration fees and gasoline taxes in 1936 were less than the total expenditures for construction and maintenance on the respective State primary systems. A definite change has occurred from the earlier policy of financing streets and highways out of general funds, and the four States included in this study have made considerable progress toward financing primary highways from highway-user revenues.

An early contention was that many nonhighway users were inequitably taxed because property taxes paid by them went to the support of roads they did not use. A more current claim is that large groups of highway users are being assessed for facilities they use only to a small degree. This claim is partly based on urban residents' contentions that a large portion of their annual vehicle travel is performed on the streets of the municipality of residence, and that they make but relatively slight use of the rural roads which are constructed and maintained from gasoline and motor-vehicle taxation.

Road-use surveys have been made to determine accurately the facts needed to answer the question: Who utilizes the existing highway facilities?

DATA GIVEN ON HIGHWAY TRAVEL, REVENUES, AND EXPENDITURES

Tables 8 to 12, inclusive, show comparable figures for the source of travel, the source of revenues, and the expenditures on each of the highway systems of the States. It can be seen in table 8 that revenues from other-than-highway-user taxes played a very important part in highway finance in Minnesota and New York. In the latter State more than 40 percent of the street and highway expenditures was made out of general revenues.

TABLE 8.—*Sources of revenues spent for roads and streets*

State	Total street and highway expenditures	Specific street and highway imposts	Street and highway expenditures from general revenues	
			Amount	Percent
Colorado	$12, 676, 300	$12, 004, 800	$671, 500	5. 3
Minnesota	55, 732, 600	39, 682, 100	16, 050, 500	28. 8
New York	215, 583, 200	[1] 127, 485, 400	88, 097, 800	40. 9
Wisconsin	61, 280, 400	53, 186, 000	8, 094, 400	13. 2

[1] $59,517,300, or 46.7 percent, was specifically levied on New York City residents.

TABLE 9.—*Specific street and highway imposts paid by rural and urban residents*

State	Paid by residents of—				Total	
	Rural areas		Incorporated places			
	Amount	Percent	Amount	Percent	Amount	Percent
Colorado	$3, 556, 400	29. 6	$8, 448, 400	70. 4	$12, 004, 800	100
Minnesota	16, 880, 000	42. 5	22, 802, 100	57. 5	39, 682, 100	100
New York	26, 051, 200	20. 4	[1] 101, 434, 200	79. 6	127, 485, 400	100
Wisconsin	23, 035, 200	43. 3	30, 150, 800	56. 7	53, 186, 000	100

[1] $59,517,300 or 58.6 percent paid by New York City residents.

TABLE 10.—*Percentage of total highway expenditures paid by rural and urban residents*

State	Paid by residents of—		Total
	Rural areas	Incorporated places	
	Percent	*Percent*	*Percent*
Colorado	27. 5	72. 5	100
Minnesota	37. 2	62. 8	100
New York	21. 5	78. 5	100
Wisconsin	46. 9	53. 1	100

TABLE 11.—*Percentage of residents in rural and urban areas in 1930*

State	Residents of—		Total
	Rural areas	Incorporated places	
	Percent	*Percent*	*Percent*
Colorado	38. 0	62. 0	100
Minnesota	37. 7	62. 3	100
New York	13. 6	[1] 86. 4	100
Wisconsin	37. 1	62. 9	100

[1] 55.1 percent of the total population resided in New York City.

In studying these figures, it must be remembered that each of the States included in this study has at least one large city whose finances and physical and economic characteristics are profoundly different from those of the rest of the State. This is most pronounced in New York. While New York City alone, not including the very populous surrounding metropolitan area, comprised 55.1 percent of the population of that State; Milwaukee had but 19.7 percent of Wisconsin's population; Minneapolis and St. Paul together had but 28.7 percent (18.1 percent and 10.6 percent, respectively) of Minnesota's population; and Denver had but 27.8 percent of Colorado's population.

This accounts for some of the striking differences in New York data. Most of the specific highway imposts paid by New York City residents were in the form of motor-vehicle and motor-fuel taxes. Most of the funds for street construction in the city came out of general funds. Consequently, with necessary street expenditures large by virtue of the very size of the city, the portion of all street and highway expenditures met out of general revenue in the State as a whole was very greatly affected by the New York City expenditures of this type. The effect of New York City on data for the entire State is apparent in all tables of this group.

TABLE 12.—*Estimated percentage of motor vehicles owned by residents of rural and urban areas*

State	Vehicles owned in—		Total
	Rural areas	Urban areas	
	Percent	*Percent*	*Percent*
Colorado	29. 5	70. 5	100
Minnesota	39. 8	60. 2	100
New York	19. 5	[1] 80. 5	100
Wisconsin	41. 8	58. 2	100

[1] 34.7 percent of the total number of vehicles were owned by residents of New York City.

The detailed fiscal studies in these States indicated that highway-user revenues are generally sufficient for financing the State highways, and that those street and highway expenditures that had to be met out of other or general revenues (shown in table 8) were generally for local roads and streets.

Several measures of benefit have been suggested for use in determining the proper allocation and assessment of taxes for the construction and maintenance of highway facilities. Different measures have been applied in various States, frequently because of expediency rather than because of any general or specific knowledge of what constitutes an equitable base for levying highway taxes. It is desirable to determine first the existing relationships that would govern the selection of an equitable base.

Table 9, which shows the amounts of revenues [5] paid by residents of rural and urban areas in these four States, indicates by comparison with the data of tables 10, 11, and 12 that—

1. No uniform relation exists between the source of specific highway imposts and the source of revenues for all highway expenditures, as distinguished between rural and urban areas. Most of the specific highway revenues came from urban residents. The relatively high percentage shown for rural residents in Minnesota results from the fact that Minnesota townships levy specific property taxes for highway purposes.[6]

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[5] Under the definitions used in these surveys, specific highway taxes are those taxes levied on motor-vehicle users, as motor-vehicle registration fees, motor-fuel taxes, and other taxes that are specifically levied and committed to highway purposes. Thus, when local property taxes, levied and collected on the basis of some budgetary schedule, are used to defray highway expenditures, the portion so used is not considered a specific highway levy unless that portion of the levy scheduled for highway purposes was definitely committed for that use and could not be used interchangeably with other funds for other governmental purposes.

[6] See PUBLIC ROADS, March 1936, page 11.

2. Only in Colorado is the percentage of rural residents in the State appreciably higher than the percentage of revenues for highway expenditures that came from rural residents. In Minnesota the percentages are approximately the same and in New York and Wisconsin the population percentages are considerably lower. The ratios between the percentage of rural residents in the total population and the percentage of total highway costs paid by rural residents in these four States were as follows:

Colorado	1.38
Minnesota	1.01
New York	.63
Wisconsin	.79

In other words, Colorado rural residents paid less for the support of their streets and highways than might have been expected as reasonable on a purely per-capita basis. The variation in the other direction was greatest in New York.

3. In all States the situs of ownership of motor vehicles corresponds somewhat closely to the source of revenues spent for highways. When ratios are set up similar to those above, we find that in each of the four States the ratios between the percentage of total motor vehicles owned in rural areas and the percentage of total highway costs paid by rural residents were as follows:

Colorado	1.07
Minnesota	1.07
New York	.91
Wisconsin	.89

These ratios indicate that rural residents in Colorado and Minnesota owned motor vehicles in excess of their contributions to highway costs.

Comparison of the tables also reveals that only in Colorado was the percentage of rural motor-vehicle ownership lower than the percentage of rural population. The ratios between the percentage of rural-owned motor vehicles in the State and the percentage of rural population were as follows:

Colorado	0.78
Minnesota	1.06
New York	1.43
Wisconsin	1.13

COMPARISONS MADE OF HIGHWAY USE AND TAXATION

All these suggested criteria of highway taxation bases, i. e., source of specific levies, distribution of population, and distribution of motor-vehicle ownership, fail to indicate what relations should exist between highway-user taxation and highway expenditures; they indicate only what the present relations are. It is therefore desirable to investigate the data available as to street and highway use and to determine (1) if such data provide a basis for establishing an equitable taxation system to furnish revenues for the street and highway system, and (2) if present revenues are derived from various population groups in proportion to their interest in and use of the various highway systems.

Pertinent data on this phase of the subject are presented in tables 13 to 21, inclusive, which show:

1. The sources of funds spent on the primary State highway systems, as derived from rural and urban residents, compared with the use of these highway systems by rural and urban residents.
2. Similar comparisons for expenditures on and the use of—
    a. County roads, which frequently comprise the secondary road system of the State.
    b. Local rural roads.
    c. City and village streets.
    d. All roads and streets within the State.

Comparisons of these data are also shown graphically in figures 1 to 5. These tables and figures illustrate the following facts disclosed by the road-use surveys in these four States:

1. Colorado and Minnesota rural residents contributed more to the primary State highway systems of their respective States than their proportionate travel on those systems; New York rural residents paid far less than their proportionate use; and Wisconsin rural residents paid almost in proportion to use. The respective ratios of expenditures to travel are: Colorado—1.33; Minnesota—1.10; New York—0.65; and Wisconsin—1.05.

The proportion of truck and bus travel was found to be approximately the same in all four States. In New York it comprised 16.3 percent of the total travel on the State primary system. Strikingly enough, a larger percentage of the total truck and bus travel on primary roads originated in rural areas than originated from New York City. The highest percentage of the total truck and bus travel in New York originating in one group of places was in the group of cities having populations between 75,000 and 400,000, while the percentage was lowest in Buffalo.

Somewhat similar conditions were noted with respect to the use of secondary roads. More than one-fourth of the secondary highway travel originating in New York City was attributable to trucks and busses.

Travel on Minnesota primary highways was very similar to New York's in that 17.3 percent was truck and bus travel; and, as in New York, of rural motor-vehicle owners' travel on primary highways, more was occasioned by trucks and busses than in the case of any other population group. Of the travel on secondary roads, 16.9 percent was occasioned by trucks and busses.

Truck and bus travel constituted a somewhat larger percentage of motor travel in Colorado than in the other States. On the primary system 24.3 percent of the total travel by Colorado vehicles was truck and bus travel; 18.4 percent of the secondary road travel was attributable to those vehicles; and for all roads and streets such travel accounted for 19 percent of the total.

2. In all four States, rural residents accounted for a greater share of the travel on the secondary roads than their relative contribution to the support of those roads. In Wisconsin the proportions were more nearly equal than in any of the other States. The condition revealed in New York was somewhat unexpected, since the secondary system consists generally of good to excellent roads. It had been thought that the system was well used and served as an important adjunct to the primary system. As a result of the disclosures of the New York survey immediate steps were taken to divert some of the primary system travel to the secondary system by marking such roads and their termini more adequately. The close relation between use and payment in Wisconsin is possibly due in part to a well-maintained and carefully planned county system, adequately marked and signed.

The respective ratios between expenditures and travel by rural residents on the secondary systems were as follows: Colorado—0.68; Minnesota—0.74; New York—0.64; Wisconsin—0.92.

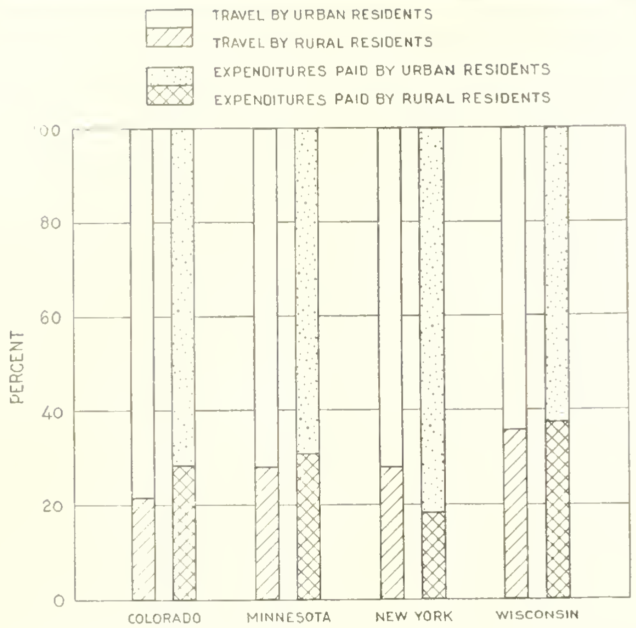

FIGURE 1.—PERCENTAGES OF TRAVEL PERFORMED AND EXPENDITURES PAID BY RESIDENTS OF URBAN AND RURAL AREAS, ON PRIMARY STATE HIGHWAYS.

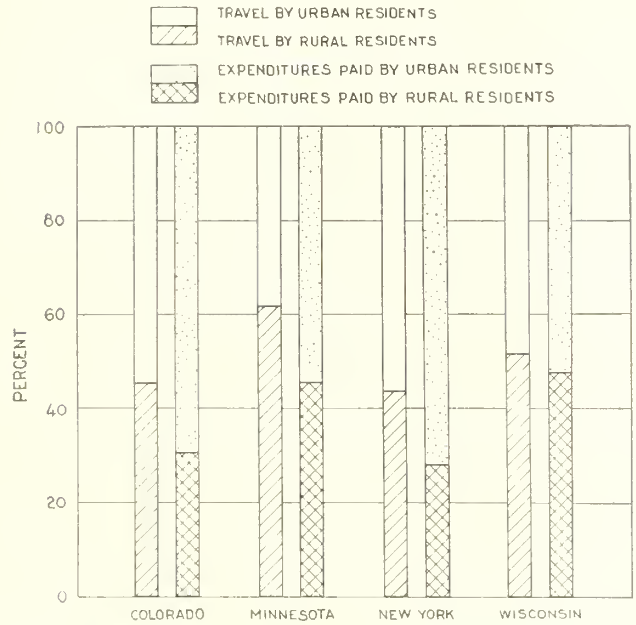

FIGURE 2.—PERCENTAGES OF TRAVEL PERFORMED AND EXPENDITURES PAID BY RESIDENTS OF URBAN AND RURAL AREAS, ON SECONDARY ROADS.

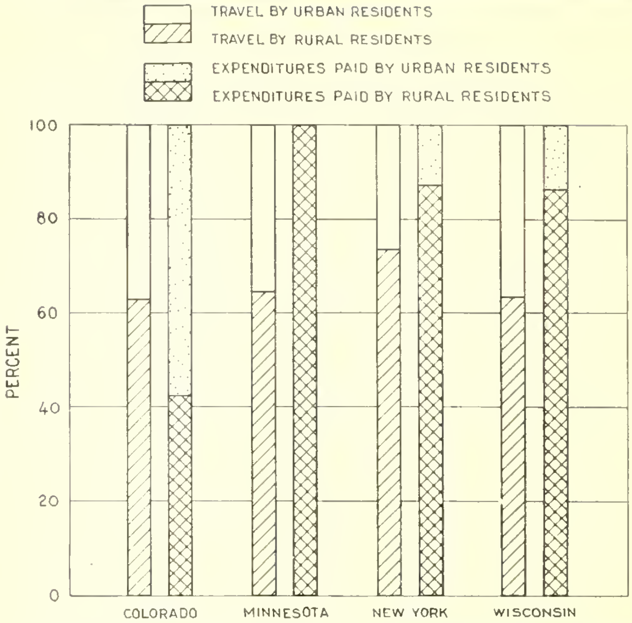

FIGURE 3.—PERCENTAGE OF TRAVEL PERFORMED AND EXPENDITURES PAID BY RESIDENTS OF URBAN AND RURAL AREAS, ON LOCAL RURAL ROADS.

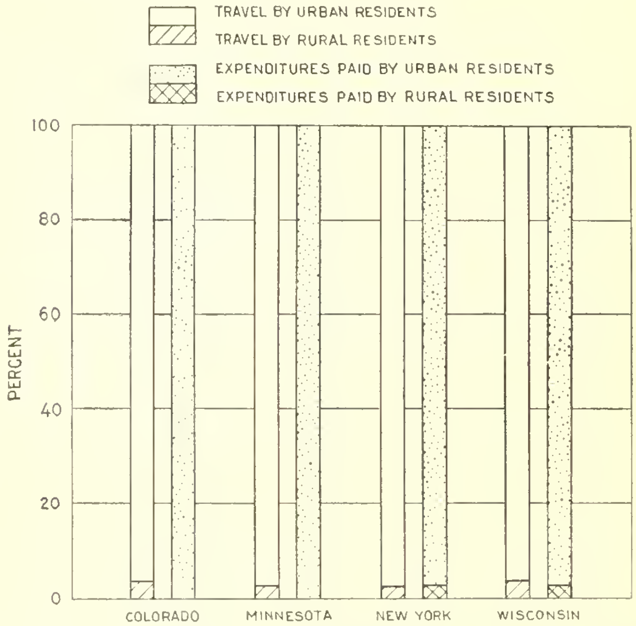

FIGURE 4.—PERCENTAGES OF TRAVEL PERFORMED AND EXPENDITURES PAID BY RESIDENTS OF URBAN AND RURAL AREAS, ON URBAN STREETS.

3. Only in Colorado did rural residents' contributions to local roads fail to outweigh their use of those roads. These local roads in Colorado are county roads, and receive the larger portion of their support from urban residents because necessary road funds are largely derived from the State motor-vehicle fees and motor-fuel taxes to which urban residents contribute a large share. In the other States the local roads are under township jurisdiction and receive the larger share of their support from township property taxes. This is notably evident in Minnesota where the township roads are entirely financed by township property taxes.

The respective ratios between expenditures and travel by rural residents on the strictly local rural roads were as follows: Colorado—0.67; Minnesota—1.55; New York—1.18; Wisconsin—1.36.

Surprisingly enough, more than one-fourth of the New York travel on township roads was by trucks and busses—but this was largely composed of light trucks which in many instances served their owners both as trucks and passenger cars. Approximately 20 percent of the travel on Minnesota's township roads was likewise performed by trucks and busses.

4. The financing and use of urban streets may be said to have been done almost entirely by urban residents. The highest percentage of rural use of urban facilities was in Wisconsin, where the survey indicated almost 8 percent of the travel on urban streets was performed by rural residents.

Comparisons of the data given in tables 13 to 19 reveal some interesting relationships. It was noted in the comments on tables 8 to 12 that these figures indicated what relationships existed but did not reveal what they should be under an equitable taxation system. In later tables comparisons between travel by rural residents and their contributions to the highways of the States have been made. It was observed that there were wide variations with respect to contributions to and use of the primary system. Colorado, Minnesota, and Wisconsin rural residents contributed more, proportionately, than they used the primary highways. In all four States rural residents contributed less, proportionately, to the secondary systems than their travel on them warranted; for the purely local rural roads, only in Colorado did rural residents contribute less, relatively, than their travel on the roads warranted.

TABLE 13.—*Percentage of expenditures on the respective State primary highway systems paid by rural and urban residents*

State	Paid by residents of—		Total
	Rural areas	Incorporated places	
	Percent	*Percent*	*Percent*
Colorado [1]	28.6	71.4	100
Minnesota	30.8	69.2	100
New York	18.2	81.8	100
Wisconsin	37.4	62.6	100

[1] Includes only the Federal-aid portion of the State highways.

TABLE 15.—*Percentage of expenditures on county roads paid by rural and urban residents*

State	Paid by residents of—		Total
	Rural areas	Incorporated places	
	Percent	*Percent*	*Percent*
Colorado [1]	30.9	69.1	100
Minnesota	45.5	54.5	100
New York	28.1	71.9	100
Wisconsin	47.6	52.4	100

[1] Consists of State highways not included in primary system; see also note 1, table 13.

The relationships between expenditures and travel on the combined highway systems will be discussed next.

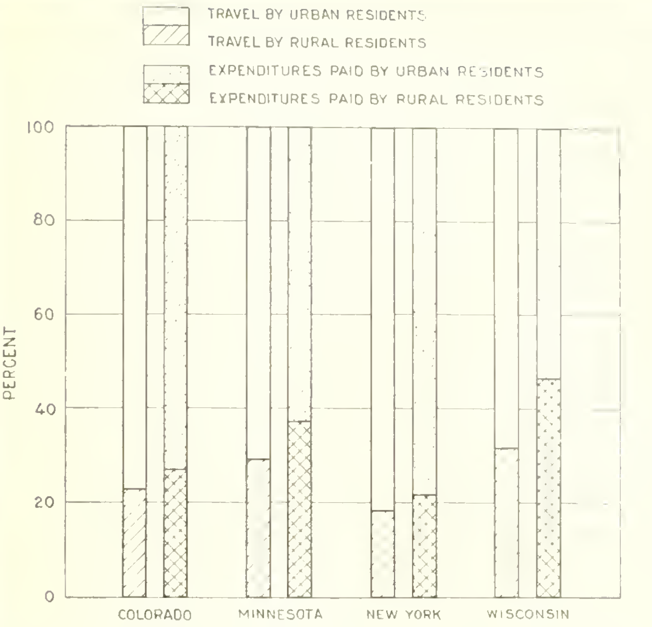

FIGURE 5.—PERCENTAGES OF TRAVEL PERFORMED AND EXPENDITURES PAID BY RESIDENTS OF URBAN AND RURAL AREAS, ON ALL STREETS AND HIGHWAYS.

TABLE 14.—*Estimated distribution of travel annually on the respective State primary highway systems*

State	Travel performed on—											
	Rural roads				Urban extensions				Entire State primary system			
	By rural residents		By urban residents		By rural residents		By urban residents		By rural residents		By urban residents	
	Million vehicle-miles	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*
Colorado [1]	178.4	21.7	648.0	78.3	29.9	20.3	106.4	79.7	208.3	21.5	754.4	78.5
Minnesota	527.2	29.8	1,243.2	70.2	110.1	21.8	395.9	78.2	637.3	28.0	1,639.1	72.0
New York	1,823.7	30.6	4,128.1	69.4	623.8	22.5	2,148.8	77.5	2,447.5	28.1	6,276.9	71.9
Wisconsin [2]	----	----	----	----	----	----	----	----	1,131.1	35.7	2,032.2	64.3

[1] See note 1, table 13.
[2] See note 2, table 3. It is impossible to separate all travel on urban streets from rural road travel.

When the data of tables 10 and 20 are compared, it is found that the ratios between the percentage of rural residents' travel to total travel and the percentage that rural residents' contributions to highways bear to the total contributions were as follows:

Colorado	0.86
Minnesota	.79
New York	.85
Wisconsin	.68

Comparing the data for rural residents in tables 12 and 20, the ratios between the percentage of travel and the percentage that rural-owned vehicles were of the total motor vehicles are found to be:

Colorado	0.80
Minnesota	.74
New York	.94
Wisconsin	.77

When each State is considered separately, it can be readily seen that in every case the contributions by rural residents are relatively greater than their use of the available highway facilities; and that relatively, their travel on the highway systems of these four States lags behind their proportionate ownership of motor-vehicles. The causes of these conditions are numerous and differ from State to State depending on the extent and nature of highway development.

TABLE 16.—*Estimated distribution of travel annually on the respective county road systems*

State	Travel performed on— Rural roads: By rural residents, Million vehicle-miles	Rural roads: By rural residents, Percent	Rural roads: By urban residents, Million vehicle-miles	Rural roads: By urban residents, Percent	Urban extensions: By rural residents, Million vehicle-miles	Urban extensions: By rural residents, Percent	Urban extensions: By urban residents, Million vehicle-miles	Urban extensions: By urban residents, Percent	Entire county road system: By rural residents, Million vehicle-miles	Entire county road system: By rural residents, Percent	Entire county road system: By urban residents, Million vehicle-miles	Entire county road system: By urban residents, Percent
Colorado[1]	111.6	46.4	128.8	53.6	7.8	35.0	14.5	65.0	119.4	45.5	143.3	54.5
Minnesota	590.1	62.1	360.7	37.9	38.2	58.1	27.5	41.9	628.3	61.8	388.2	38.2
New York	426.9	53.2	374.9	46.8	74.6	21.6	270.7	78.4	501.5	43.7	645.6	56.3
Wisconsin[2]									419.8	51.6	393.7	48.4

[1] See note 1, table 15.

[2] See note 2, table 3. It is impossible to separate all travel on urban streets from rural road travel.

TABLE 17.—*Percentage of expenditures on local rural roads paid by rural and urban residents*

State	Paid by residents of— Rural areas	Paid by residents of— Incorporated places	Total
	Percent	*Percent*	*Percent*
Colorado[1]	42.5	57.5	100
Minnesota	100.0	0	100
New York	[2] 87.2	[2] 12.8	100
Wisconsin	86.2	13.8	100

[1] County roads; in all other States these are township roads.

[2] Because of fund relationships involved and the overlapping of the functions of the towns and the incorporated villages, it is impractical to determine the true contributions; but it would appear that at least 13 percent of the cost of town highway construction and maintenance was borne by the residents of New York cities and villages.

TABLE 18.—*Percentage of expenditures on urban streets paid by rural and urban residents*

State	Paid by residents of— Rural areas	Paid by residents of— Incorporated places	Total
	Percent	*Percent*	*Percent*
Colorado	0.0	100.0	100
Minnesota	0	100.0	100
New York	[1] 2.9	[1] 97.1	100
Wisconsin	2.7	97.3	100

[1] It is impractical to determine the exact rural and urban contributions to urban streets, but on the basis of the payment of or use of State funds for urban streets it would appear that approximately these percentages of the costs of urban streets were contributed by the two classes of residents.

CLOSE CORRELATION FOUND BETWEEN REGISTRATIONS, EXPENDITURES, AND TRAVEL PER MILE OF HIGHWAY

It is believed that the data shown here illustrate clearly the existing conditions and point the way toward remedies that might be effected. Undoubtedly, thorough investigation along several lines will be necessary to discover the causal factors behind the ratios appearing above, and to work out the remedial measures that should be taken.

These comparisons are also clearly shown in figures 1 to 5, inclusive. The bar diagrams illustrate the uniformity of relationships in the four States with the following exceptions:

1. The primary system in New York.
2. The local rural roads in Colorado.
3. Urban streets in Wisconsin.

A comparison of the travel on each of the systems and the actual mileage of those roads will also be made. Comparisons of tables 2 and 21 indicate: (1) That the percentage of travel on the primary systems is much higher in all States than the proportionate lengths of those systems; (2) that these percentages are more nearly equal for the secondary roads; and (3) that the widest discrepancy occurs in the case of local rural roads and urban streets. In Colorado the local (county) roads carry 8.9 percent of the travel and comprise 76.7 percent of the street and highway mileage of the State, while the city and town streets carry 28.8 percent of the travel and account for only 6.3 percent of the total road and street mileage.

Certain travel data are available for five other States in which financial surveys were made without the accompanying road-use surveys.[7] Questionnaires returned by motor-vehicle owners in these States provided data as to the total annual travel by motor-vehicle owners on the streets and roads of the States of which they were resident.

In figure 6 some comparisons of the data obtained in these nine States are presented. The number of vehicles per mile of road, expenditures per mile of road,

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[7] These States and the years for which the surveys were made are: Michigan, 1930; Illinois, 1930; New Hampshire, 1932; Wyoming, 1932; and New Mexico, 1932.

TABLE 19.—*Estimated distribution of travel annually on the respective local rural roads and urban streets*

State	Travel performed on— Local rural roads: By rural residents, Million vehicle-miles	Local rural roads: By rural residents, Percent	Local rural roads: By urban residents, Million vehicle-miles	Local rural roads: By urban residents, Percent	Urban streets: By rural residents, Million vehicle-miles	Urban streets: By rural residents, Percent	Urban streets: By urban residents, Million vehicle-miles	Urban streets: By urban residents, Percent	All local roads and streets: By rural residents, Million vehicle-miles	All local roads and streets: By rural residents, Percent	All local roads and streets: By urban residents, Million vehicle-miles	All local roads and streets: By urban residents, Percent
Colorado	110.2	63.1	64.7	36.9	21.0	3.8	546.5	96.2	131.2	17.7	611.2	82.3
Minnesota	201.3	64.7	109.9	35.3	43.2	2.8	1,484.4	97.2	244.5	13.3	1,594.3	86.7
New York	372.3	73.8	132.5	26.2	[1] 232.5	2.6	[2] 8,863.9	97.4	604.8	6.3	8,996.4	93.7
Wisconsin	301.8	63.5	173.3	36.5	[3] 58.8	3.8	[3] 1,486.3	96.2	360.6	17.8	1,659.6	82.2

[1] 2,200,000 vehicle-miles on New York City streets.

[2] 5,402,300,000 vehicle-miles on New York City streets.

[3] This does not include street travel on the primary and secondary system where the connecting portions in urban areas were used.

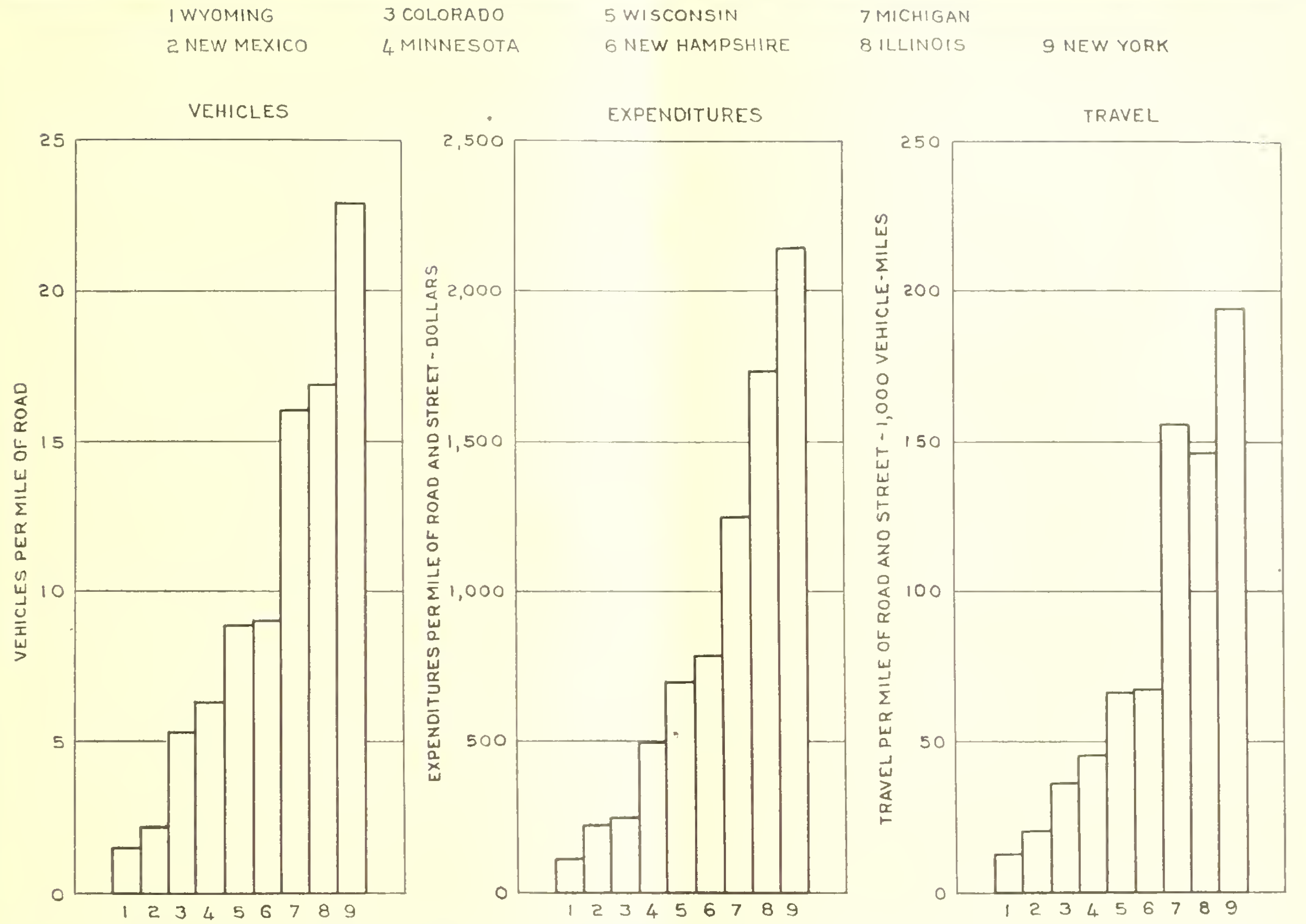

FIGURE 6.—COMPARISON OF VEHICLES PER MILE, EXPENDITURES PER MILE, AND TRAVEL PER MILE, ON ALL ROADS AND STREETS IN NINE STATES.

TABLE 20.—*Estimated distribution of travel annually on all streets and highways*

State	Travel performed on—											
	Rural roads				Urban streets				All streets and highways			
	By rural residents		By urban residents		By rural residents		By urban residents		By rural residents		By urban residents	
	Million vehicle-miles	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*	*Million vehicle-miles*	*Percent*
Colorado	400.2	32.2	841.5	67.8	58.7	8.1	667.4	91.9	458.9	23.3	1,508.9	76.7
Minnesota	1,318.6	43.5	1,713.8	56.5	191.5	9.1	1,907.8	90.9	1,510.1	29.4	3,621.6	70.6
New York	2,622.9	36.1	4,635.5	63.9	[1] 930.9	7.6	[2] 11,283.4	92.4	3,553.8	18.3	15,918.9	81.7
Wisconsin [3]									1,911.5	31.9	4,085.5	68.1

[1] 16,400,000 vehicle-miles on New York City streets.
[2] 6,019,200,000 vehicle-miles on New York City streets.
[3] See note 2 table 3. It is impossible to segregate all travel on urban streets from rural road travel.

and vehicle-miles of travel per mile of road for each State are shown. In each of these three comparisons, all of the States bear the same general relationship to each other except Illinois, where the total travel per mile of road was lower than that of the preceding State (Michigan).

Measurable increases in the total travel on the various road systems have occurred since these studies were made, as evidenced both by increased annual travel per vehicle (as revealed by gasoline consumption data) and by an increase in the number of registered vehicles. The State-wide highway planning surveys now in progress in these 9 States and in 34 other States will reveal these changes and will indicate whether or not these increases have altered the relationships shown in figure 6.

Figure 7 shows in a different manner the same relationships as figure 6 for the four States Colorado, Minnesota, New York, and Wisconsin. This figure further illustrates that a State having a high expenditure per mile of road and street also has a large number of vehicles per mile of road, and also has a correspondingly great amount of travel per mile of road.

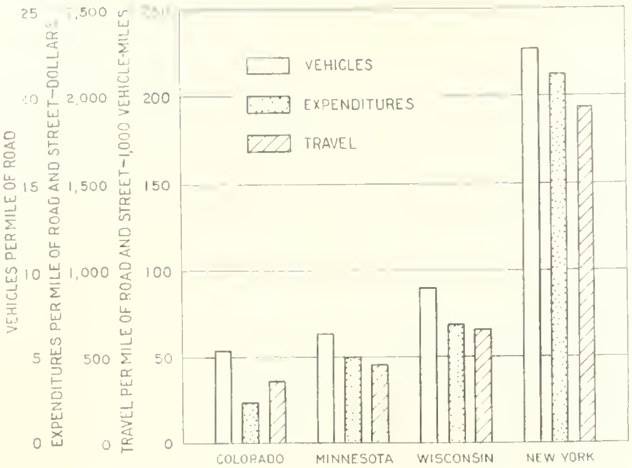

Figure 7.—Comparison of Vehicles per Mile, Expenditures per Mile, and Travel per Mile on all Roads and Streets in Four States.

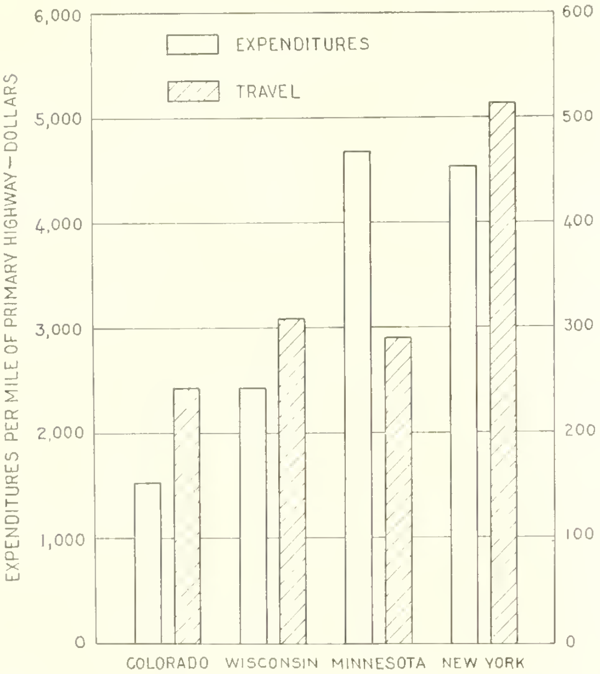

Figure 8.—Comparison of Expenditures per Mile and Travel per Mile on the Primary Rural Highways in Four States. (Travel Figures for Wisconsin are in Vehicle-Miles per Mile of all Primary Highways.)

The number of vehicles per mile of road in Colorado was 5.4 and expenditures per mile were $241, while in Wisconsin with 8.9 vehicles per mile of road, or 65 percent more than in Colorado, expenditures were $684 per mile, or 184 percent greater than for Colorado.

Corresponding figures are observed in connection with travel per mile of road. In Colorado, the annual travel of 37,400 vehicle-miles per mile of road was accompanied by an annual expenditure of $241 per mile; whereas in Minnesota, greater travel, 46,000 vehicle-miles per mile of road, accompanied expenditures of $500 per mile of road. Thus, travel in Minnesota was but 23 percent greater than in Colorado, yet the former's expenditures were 107 percent greater, when compared on this basis.

Figures 8 to 11, inclusive, illustrate in detail the relations existing between travel per mile of road and expenditures per mile of road for each of the street and highway systems in these four States.

Although these relationships follow similar trends, figure 8 indicates that for Minnesota, by comparison with the other three States, the travel per mile on the rural portions of the primary State highways did not correspond with the large unit expenditures.

Table 21.—*Percentage distribution of total road and street mileage in each State*

State	Mileage of—				Total
	Primary rural roads [1]	Secondary rural roads [2]	Local rural roads [3]	Urban streets	
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Colorado	6.5	10.5	76.7	6.3	100
Minnesota	5.4	28.2	65.1	[4] 1.3	100
New York	11.4	11.6	59.7	[5] 17.3	100
Wisconsin	11.4	15.4	65.8	7.4	100

[1] State highways except in Colorado where only Federal-aid roads are included.

[2] County roads except in Colorado where State roads not included in the primary system are so classed.

[3] Township roads except in Colorado where this group consists of county and forest roads.

[4] Percentages computed without including most street mileage, for which data were not available. The amount shown here represents the streets that formed the urban connections of the State and county highways.

[5] 5.3 percent of New York mileage was comprised of New York City streets.

The ratios shown for Colorado and New York are surprisingly similar. New York expenditures per mile of primary rural road were 197 percent greater than those for Colorado, whereas New York travel expressed in vehicle-miles per mile of primary rural road was approximately 113 percent greater. Comparisons with Wisconsin data are not entirely satisfactory because travel on urban and rural portions of the system cannot be separated.

A similar condition to that noted for Minnesota in figure 8 is seen to have existed for the rural portions of county roads in New York (fig. 9). Expenditures per mile of these roads were eight times greater than those reported in Colorado, and seven and one-quarter times greater than those in Minnesota, but travel was only about 58 percent greater than travel on Colorado secondary roads and 76 percent greater than that in Minnesota.

EXPENDITURES PER VEHICLE-MILE OF TRAVEL LEAST ON URBAN STREETS

The uniformities noted for the entire road network within these States (fig. 7) and for the primary systems (fig. 8) do not extend so markedly to the secondary roads. The greater variations for secondary roads are undoubtedly caused largely by the fact that the development, function, and use of the secondary roads are more divergent than for the primary roads or even for the local roads and streets.

Primary highways generally serve as trunk-line transportation systems; local rural roads usually receive their greatest utilization from the residents living adjacent to them, just as the principal function served by city and village streets is the handling of local traffic. Secondary roads are generally integral accessories of the primary roads; sometimes they serve more generally as primary local roads; and occasionally, laid out without rational plan or purpose, they merely exist, serving various needs. It is not surprising then that the conditions shown in figure 9 are found; nor is it surprising that these nonuniform conditions affect the relationships for all roads and streets but very little, since the travel and expenditures on the secondary systems generally constitute but a small percentage of the total highway travel and expenditures in the State. In these four States, the travel on the secondary systems constituted the following percentages of the total travel within the States:

	Percent
Colorado	13. 4
Minnesota	19. 8
New York	5. 9
Wisconsin	13. 6

Expenditures on the secondary systems constituted the following percentages of the total street and highway expenditures within the respective States:

	Percent
Colorado	16. 3
Minnesota	22. 8
New York	17. 8
Wisconsin	15. 4

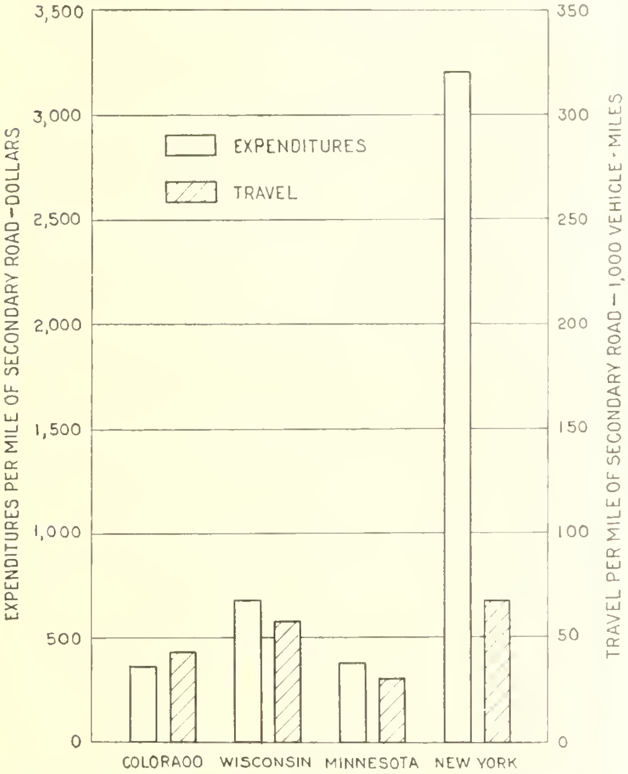

FIGURE 9.—COMPARISON OF EXPENDITURES PER MILE AND TRAVEL PER MILE ON SECONDARY RURAL ROADS IN FOUR STATES. (TRAVEL FIGURES FOR WISCONSIN ARE IN VEHICLE-MILES PER MILE OF ALL SECONDARY ROADS.)

The differences shown in these figures are more strikingly illustrated by figure 9 where travel and expenditures for the rural portions of the systems are shown.

Great variation is also noted in the township road relationships, shown graphically in figure 10. Though there is but comparatively little variation in travel per mile of local road in the four States, expenditures per mile of road vary from $54 in Minnesota to $375 in New York.

Street mileages in Minnesota were not available so figure 11 shows data for only three States.

Another comparison for these four States can be made by showing the expenditures per vehicle-mile of travel on each of the systems. These data are given in table 22.

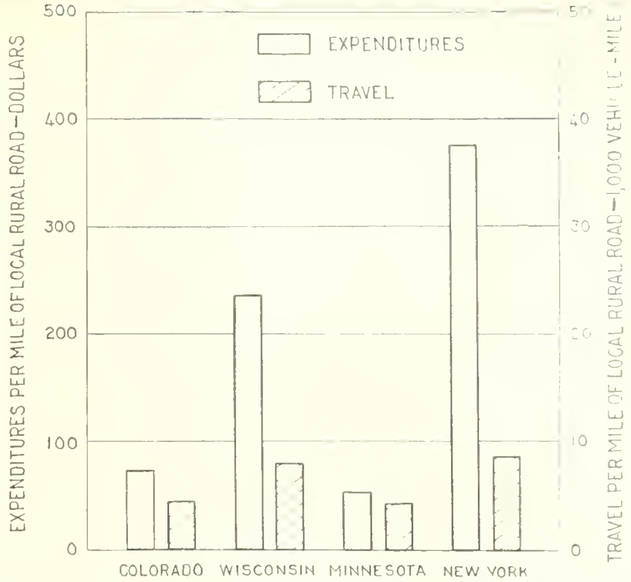

FIGURE 10.—COMPARISON OF EXPENDITURES PER MILE AND TRAVEL PER MILE ON LOCAL RURAL ROADS IN FOUR STATES.

TABLE 22.—*Expenditures per vehicle-mile of travel on the various road systems*

State	Expenditures per vehicle-mile of travel on—				
	Primary rural roads	Secondary rural roads	Local rural roads	Urban streets	All roads and streets
Colorado	$0. 0063	$0. 0082	$0. 0035	$0. 0035	$0. 0064
Minnesota	. 0161	. 0128	. 0125	. 0053	. 0109
New York	. 0088	. 0469	. 0446	. 0084	. 0111
Wisconsin	. 0220	. 0225	. 0386	. 0032	. 0102

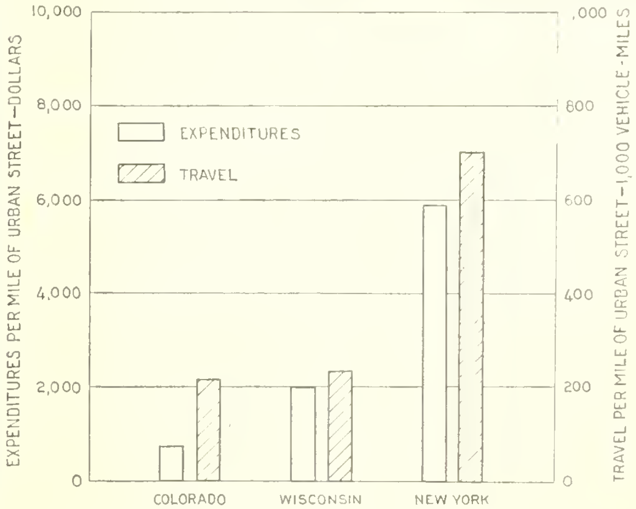

FIGURE 11.—COMPARISON OF EXPENDITURES PER MILE AND TRAVEL PER MILE ON URBAN STREETS IN THREE STATES.

Colorado had the lowest expenditure per vehicle-mile of travel, and the highest costs were for New York secondary and local roads and for Wisconsin township roads. In all States, urban street costs were relatively low, those in New York being the highest. This is not unexpected, as extreme urbanization tends to nullify some of the expected economies by requiring additional expensive facilities.

Before any conclusions are drawn from these data, certain other specific studies that also reveal information of considerable interest should be made.

The four States of this analysis contain three cities having approximately the same population—Buffalo, Milwaukee, and Minneapolis. It is therefore interesting to study the similarity of travel habits of passenger car owners in these cities as revealed by the following data:

The estimated average annual State travel of passenger cars owned in these cities was—

	Miles
Buffalo	9, 100
Milwaukee	9, 400
Minneapolis	9, 800

The approximate percentage of the total travel that was performed on the primary State highways by passenger cars was—

	Percent
Buffalo	36. 4
Milwaukee	37. 0
Minneapolis	37. 1

The number of persons per motor vehicle owned in these three cities also provides an interesting comparison with the first tabulation given above. There appears to be close correlation between these two figures in that the higher average annual travel was reported for the city that had the fewest number of persons per vehicle (Minneapolis). The figures are as follows:

	Persons per vehicle
Buffalo	4. 5
Milwaukee	4. 2
Minneapolis	3. 8

It is also possible to compare percentages of travel performed on rural and urban roads by motor-vehicle owners in various-sized cities of these States. The data are shown in table 23.[8]

It will be observed that the groups of cities have been arranged in ascending order of population and that there is a general tendency for the percentage of total travel on rural roads to decrease as the size of city increases. While the trend is not constant, it is quite evident and discrepancies in most cases can be explained by virtue of the location of the respective cities and the peculiarities of the highway systems serving the cities.

TABLE 23.—*Approximate percentage distribution of rural and urban travel by passenger-car owners of selected cities*

State	City	Population (1930)	Percentage of travel on—		Total
			Rural roads	Urban streets	
Minnesota	Hibbing Rochester St. Cloud Winona	15, 666 20, 621 21, 000 20, 850	55	45	100
Colorado	Colorado Springs Pueblo	33, 237 50, 096	45	55	100
New York	All places from 15,000 to 75,000.		43	57	100
Minnesota	Duluth St. Paul	101, 463 271, 606	25	75	100
New York	All places over 75,000 except Buffalo and New York City.		38	62	100
Colorado	Denver	287, 861	28	72	100
Minnesota	Minneapolis	464, 356	27	73	100
Wisconsin	Milwaukee	578, 249	35	65	100
New York	Buffalo	573, 076	30	70	100
	New York City	6, 930, 446	8	92	100

Similar studies have been made of the distribution of travel performed on the other highway systems of the States by cars owned in these cities. Characteristics similar to those observed in table 23 are plainly discernible.

Study of the travel in various cities indicates that but a small percentage of local travel in large cities comes from outside the city. This is shown in table 24. In each of these large cities well over 80 percent of the total travel on their streets is accounted for by vehicles owned in those cities.

TABLE 24.—*Approximate source of travel originating within the respective States and performed on the streets of selected cities*

Travel originating in—	Percentage of total State travel performed on the streets of—				
	Denver	Minneapolis	St. Paul and Duluth	Buffalo	New York City
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Rural areas	5	1	4	5	2
Places under 2,500	3	2	1	1	(1)
Places 2,500–15,000	4	2	3	3	3
Places 15,000–75,000	1	1	(1)	6	1
Places 75,000–400,000	87	2	83	2	1
Places 400,000–1,000,000		92	9	82	(1)
New York City				1	93
Total	100	100	100	100	100

[1] Less than 0.5 percent.

BASIS FOR EQUITABLE HIGHWAY TAXATION NEEDED

It appears logical to assume that when data from other States become available it will confirm the indications of the present studies. In any event the data will make it possible to establish more definitely the factors controlling the source and distribution of travel within a State by its resident motor-vehicle owners.

Equally interesting is the summary, shown in table 25, of data for several cities arranged in descending order of percentage of population of the State. The relative contributions to travel and highway revenues are indicated for each city or group of cities. The column showing the percentage of total State population represented by each city or group has been included to emphasize the remarkably close correlation with columns 4 and 5.

It will be seen that with the exception of a few cities, the trend of relative contribution to travel on the State highway system follows the population trend of the cities quite closely. The most pronounced difference is in the case of New York City where the picture is distorted because of the fact that so much of New Yorker's travel outside New York City is on the highways of other States. Unless the New Yorker goes north along the Hudson River, he soon crosses the line into adjacent States.

Correlation between population and fiscal contribution by the residents of these cities to the State highway system is extremely close. This type of tabulation seems particularly valuable in setting forth the relationships between particular cities of population groups. Other bases than that of population might well have been selected, preferably motor-vehicle ownership distribution, but the population base appears adequate for showing the existing relationships.

These observations indicate that it may be possible to develop factors with respect to the influences of population, vehicle concentration, geographical location, economic conditions, and extent of the highway system, on the basis of which reliable estimates may be made regularly as to the use to which the highway systems are being put. It is essential that such factors be developed if an equitable taxation system is not only to be established but also maintained. The data shown herewith indicate the relationships that appear to exist, the existing sources of data for establishing equitable tax systems, and the necessary factors that need to be developed.

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[8] Data for Wisconsin are not available by individual cities except for Milwaukee.

TABLE 25.—*Comparison of approximate contributions of residents of selected cities to the financial support of their State highway systems and the percentage of their travel occurring on those systems*

State	City	Percentage of total population in State in 1930	Percentage of total annual travel on State highways performed by residents of these cities	Percentage of total annual contributions to State highways paid by residents of these cities	Percentage of total annual State travel by residents of these cities that was on State highways	Percentage of total annual highway contributions by residents of these cities that was expended on State highways
New York	New York	55.1	12	35	16	20
Colorado	Denver	27.8	24	31	36	32
Wisconsin	Milwaukee	19.7	14	19	35	46
Minnesota	Minneapolis	18.1	19	22	36	44
	St. Paul Duluth	14.6	10	15	32	40
New York	Cities 75,000–400,000.	9.1	13	12	48	22
	Cities 15,000–75,000.	8.0	14	11	54	24
Colorado	Pueblo Colorado Springs	8.0	11	8	52	33
New York	Buffalo	4.5	5	6	34	24
Minnesota	Cities 15,000–75,000.	3.1	4	3	47	27

It is probable that the relationships developed in the surveys in these four States have changed somewhat since the studies were made. It is believed, however, that these relationships have remained fairly constant, though total travel may vary considerably. Further research into the validity of the relationships discussed in this article may be an important step in the establishment of an equitable highway taxation program within any given State.

Bearing in mind the various factors and data described above, and realizing that it might be possible to establish a highway development and finance program on the basis of adequate traffic, inventory, and financial-road-use surveys, it should be remembered that no equitable taxation system can be set up with only the factors discussed above as controls. The determination of an equitable basis of taxation for any purpose or for any group of the State involves a thorough study of many factors.

It might be contended that, because the residents of a given city may account for 30 percent of the total travel on a given highway system, they should not contribute more than 30 percent of the funds necessary for the upkeep of that system. While this analysis points out the relationships that exist, it should not be inferred that these considerations alone are adequate for the establishment of a sound highway fiscal policy.

Every unit of society within the State is benefited by the highway system in ways not satisfactorily measurable by present methods. For example, the maintenance of good roads within a fertile agricultural area may make possible the marketing of crops and produce in urban centers that would not otherwise be readily accessible. Thus, land that might be utilized only as a subsistence producer because of inaccessibility of markets becomes a considerable wealth producer when those markets are made available.

In a similar manner it is impossible to measure the value of the services that urban communities receive from roads serving them. While a city may, on the basis of road use alone, account for only 25 percent of the travel on the highway system, and may contribute 35 percent of the funds used on that system, as yet no method has been devised to measure the benefits derived because the highway has made the city accessible.

Several cities might be selected in the States studied where a good highway system has greatly facilitated the development of the city as a distributing or marketing center. Highway systems have led to the development of new trading areas and have immeasurably benefited the cities or centers so affected.

The value of streets within cities cannot be measured solely by the traffic using them. Even more than in the case of rural roads, they serve as a means of access to property, as essentials to adequate fire protection, as aids to health and sanitation, as thoroughfares for utility services, and in very crowded areas as sources of light and air to the adjacent property. Consequently, it appears justifiable that a substantial percentage of urban street expenditures should be met out of property taxation.

Road-use surveys provide a means of evaluating one of the most important factors in establishing the proper bases of rural and urban taxation for highway purposes. It would be highly desirable to establish additional means of evaluating other benefits received by virtue of the construction and maintenance of our present highway systems. Until satisfactory means can be developed, the most nearly equitable approach appears to be that which evaluates one of the most important single factors in determining the basis of highway finance—the actual use of the existing highway systems.

CONCLUSIONS

The data presented in this analysis summarize the available information on sources of vehicular travel in four States, make possible a few more or less general conclusions, and suggest the direction of further study necessary for the correct determination of a satisfactory highway financing program.

1. Road-use surveys provide suitable and necessary information as to the source and distribution of travel on the highway systems of a State.

2. The proportion of funds for highway purposes paid by rural and urban residents is not closely correlated with the distribution of population in such areas; the proportion of funds paid, however, is somewhat closely allied to vehicle ownership.

3. In a comparison of the four States studied, there is a direct relationship between the density of registration of motor vehicles, the volume of annual travel, and the average annual expenditures per mile of road.

4. Little relation exists between the relative use of the respective highway systems by rural and urban residents and the contributions of those residents for the support of the highways. The closest relationships appear to exist between the use and financing of State primary highways.

5. Use of the various road systems by urban residents appears to vary directly with the location and size of the incorporated place in which the travel originates.

a. Urban residents' travel outside their cities of domicile is almost entirely on the State primary systems

b. The secondary and local rural road systems are largely used by those vehicle-owners resident on such routes.

c. A very small percentage of the total traffic on city streets originates outside the city, though this percentage appears to increase as the size of the city decreases.

d. The proportion of urban residents' travel on the primary system decreases as the size of the place increases.

6. The relation of travel distribution to the situs of motor-vehicle ownership and size of the places or residence of motor-vehicle owners is partially affected by factors of geographical location, condition of the highway system, economic conditions of the community, etc., whose effects are as yet unmeasured.

7. Until other factors can be ascertained the results of the road-use surveys provide the sole criterion upon which equitable and adequate highway finance programs can be established.

The questions propounded in the first part of this analysis cannot be entirely answered by road-use surveys, but much factual data can be obtained to provide a working basis for highway tax legislation until complete data are made available.

BIBLIOGRAPHY ON HIGHWAY SAFETY NOW AVAILABLE

A selective bibliography on highway safety has recently been published by the Bureau of Public Roads of the United States Department of Agriculture as Miscellaneous Publication No. 296.

The bibliography includes references to books, articles printed in technical and other priodicals, and publications of societies. It covers the period from 1928 through May 1937. Since 1928 traffic conditions have changed so rapidly that prior publications are chiefly of historical value.

Librarians, students, and research workers will find this publication an important aid in locating published material on highway safety.

Single copies of Miscellaneous Publication No. 296 can be obtained, without charge, from the Division of Publications, United States Department of Agriculture, Washington, D. C.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MARCH 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			* BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,122,820	$ 551,410	64.3	$ 5,550,020	$ 2,774,155	260.4	$ 736,910	$ 368,450	18.1	$ 6,753,330
Arizona	1,817,080	1,305,224	91.2	1,644,059	1,159,568	79.4	564,311	333,044	35.4	2,107,972
Arkansas	2,854,540	2,835,323	176.0	1,281,994	1,274,137	85.9	102,035	101,707	4.6	4,320,542
California	6,800,621	3,745,997	150.1	7,589,123	3,990,175	126.7	2,614,921	1,396,135	64.3	4,520,149
Colorado	3,176,286	1,767,031	119.8	1,687,061	926,222	56.4	609,769	328,878	16.9	3,638,845
Connecticut	783,198	389,196	9.3	231,520	108,123	.8	314,610	157,095	3.0	2,142,456
Delaware	467,663	229,797	18.2	70,588	34,105	2.8	645,849	295,161	14.3	1,543,324
Florida	614,637	305,369	21.2	2,822,342	1,411,171	58.3	437,000	218,500	7.7	4,151,640
Georgia	2,110,923	1,020,343	122.5	6,507,517	3,253,350	298.9	955,400	477,700	46.4	7,036,783
Idaho	2,606,681	1,530,005	198.9	1,068,560	636,604	85.3	607,199	360,814	21.5	1,983,721
Illinois	10,478,221	5,126,123	307.4	8,153,961	4,008,249	167.8	3,992,650	1,996,250	72.9	4,921,201
Indiana	5,688,737	2,813,518	132.2	3,469,691	1,734,845	105.9	1,475,460	737,730	31.9	4,427,779
Iowa	6,688,709	3,058,796	219.3	4,961,646	2,241,356	143.6	1,477,728	689,150	44.1	3,305,441
Kansas	4,657,558	2,305,827	262.3	2,758,336	1,379,069	47.5	2,419,907	1,209,948	104.3	5,720,622
Kentucky	2,386,708	1,189,257	64.8	3,521,394	1,760,697	107.6	3,839,396	1,915,699	147.4	3,118,418
Louisiana	582,530	286,501	15.0	6,352,674	1,329,105	39.5	6,869,666	1,400,545	50.1	2,982,278
Maine	1,869,331	934,382	50.7	2,018,226	1,009,113	44.4	445,660	222,825	11.4	1,204,598
Maryland	930,330	465,145	13.4	1,814,594	905,534	28.9	460,360	230,180	.5	2,512,396
Massachusetts	4,419,633	2,209,815	20.3	1,911,053	955,526	3.5	670,950	335,330	5.2	3,313,625
Michigan	6,485,843	3,181,092	166.9	5,862,868	2,931,434	135.7	1,304,020	607,710	14.8	3,785,815
Minnesota	6,428,566	3,191,813	312.4	3,517,563	1,747,537	152.4	1,625,106	802,448	79.9	4,245,715
Mississippi	2,789,501	1,394,651	138.4	4,023,190	2,010,890	181.2	1,704,720	852,310	68.2	4,572,251
Missouri	8,217,741	4,023,184	443.6	5,063,704	2,451,624	140.6	3,257,601	1,379,292	86.7	4,770,139
Montana	3,824,816	2,144,049	277.7	1,996,529	1,122,671	83.9	419,566	236,000	33.0	4,547,601
Nebraska	2,457,556	1,208,532	255.9	5,115,485	2,544,926	455.0	3,743,014	1,241,194	216.3	3,728,120
Nevada	2,515,471	2,159,343	132.5	708,345	614,170	62.5	341,799	296,400	13.6	2,053,915
New Hampshire	360,696	177,963	6.5	541,078	267,567	11.6	843,979	365,921	11.4	1,234,845
New Jersey	2,055,958	950,684	20.3	2,487,960	1,243,610	17.1	274,716	136,523	2.8	2,817,790
New Mexico	3,729,069	2,286,937	260.9	1,993,134	1,315,634	97.0	464,692	283,294	73.0	1,975,873
New York	14,122,720	6,625,591	245.5	12,049,354	5,978,160	210.3	2,163,690	1,080,345	32.1	6,467,239
North Carolina	4,930,582	2,458,637	374.2	6,185,050	2,921,225	252.0	879,060	422,960	49.5	4,255,487
North Dakota	971,668	963,709	186.4	1,270,301	1,249,791	77.4	1,607,560	1,470,870	102.4	3,977,710
Ohio	4,388,541	2,119,212	56.7	8,839,122	4,389,115	98.5	1,644,123	822,920	16.3	9,290,464
Oklahoma	3,455,089	1,800,628	170.4	2,923,214	1,483,197	104.4	2,212,415	1,174,092	109.6	5,127,170
Oregon	3,979,127	2,342,471	130.9	1,478,085	901,439	81.1	514,274	313,876	14.9	2,779,439
Pennsylvania	12,898,222	6,407,944	176.3	6,818,788	3,392,103	102.2	2,729,711	1,363,818	55.6	7,009,328
Rhode Island	912,883	447,633	8.1	796,330	398,165	10.8	454,760	227,380	3.0	1,241,096
South Carolina	3,766,566	1,590,753	260.9	5,149,594	2,184,743	239.7	1,039,331	469,490	43.9	2,282,927
South Dakota	2,391,402	1,352,149	246.1	1,955,003	1,081,040	186.4	1,490,310	837,010	93.4	4,306,391
Tennessee	1,827,160	910,804	69.5	2,281,404	1,140,702	79.8	2,512,840	1,256,340	72.3	5,852,952
Texas	12,811,762	6,386,664	849.6	11,948,354	5,932,772	660.3	2,889,851	1,376,195	159.8	10,544,912
Utah	1,349,100	954,211	134.1	751,500	537,038	57.4	369,770	265,054	8.7	2,258,882
Vermont	1,052,720	505,707	29.6	1,679,730	739,095	43.1	6,730	3,169		670,927
Virginia	2,894,654	1,436,704	137.7	5,153,613	2,536,130	139.2	1,243,190	597,619	37.2	2,699,279
Washington	2,005,875	1,036,705	74.2	3,649,983	1,915,812	46.1	312,326	163,400	5.4	2,334,617
West Virginia	1,476,181	741,784	42.1	1,342,447	819,193	39.5	608,427	459,750	9.5	3,030,354
Wisconsin	8,232,929	3,976,402	273.6	4,701,047	2,159,177	107.5	854,392	400,300	13.2	3,831,229
Wyoming	2,502,655	1,485,043	263.2	1,586,299	964,116	160.5	151,720	93,650	7.9	1,629,495
District of Columbia	-	-	-	-	-	-	-	-	-	-
Hawaii	678,411	328,796	12.5	627,330	310,380	10.1	691,600	345,340	12.7	1,453,067
Puerto Rico				1,075,539	535,790	20.3				698,585
TOTALS	185,569,670	96,658,854	7,813.6	176,986,302	88,710,350	5,807.2	67,595,074	32,119,811	2,147.1	185,178,734

*Includes funds apportioned for the fiscal year 1939

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MARCH 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS (*)
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 209,700	$ 104,850	11.9				$ 948,122
Arizona	$ 36,892	$ 25,200	2.4	7,624	5,489		$ 261,190	$ 137,267	18.2	555,231
Arkansas							13,126	6,563		857,545
California	4,899	2,843	3.2	466,208	268,102	19.0	261,130	151,568	50.8	1,496,185
Colorado				234,586	129,548	11.7	180,550	100,390	7.4	691,630
Connecticut										318,278
Delaware										246,875
Florida				10,106	5,053					669,799
Georgia				137,178	66,377	13.2	258,160	126,960	32.7	1,084,289
Idaho	165,150	81,383	26.3	247,245	111,953	10.5	119,723	59,841	14.5	368,596
Illinois				382,532	191,266	16.1	1,178,400	589,200	101.4	1,286,349
Indiana							263,288	68,800	23.3	1,173,638
Iowa										1,298,449
Kansas	15,224	7,612	17.9	9,850	4,925	6.4	82,500	41,250	1.0	1,277,674
Kentucky	184,383	92,077	72.2	594,800	186,854	89.4	197,605	58,828	22.5	591,918
Louisiana										724,136
Maine	40,550	20,275	2.9	330,936	165,468	21.0	46,400	23,200	2.5	233,463
Maryland				6,264	3,132					409,344
Massachusetts				5,300	2,650					696,530
Michigan										1,535,539
Minnesota				163,702	81,851	3.2	222,760	81,868	42.3	1,215,861
Mississippi										888,927
Missouri	849,923	412,284	223.3	310,340	152,155	52.0	266,880	84,845	22.8	881,280
Montana				13,983	7,865					1,027,170
Nebraska	40,394	20,197	1.1	138,306	69,153	20.9	358,394	179,197	56.5	775,561
Nevada	60,933	52,493	9.1	294,144	254,800	32.9	13,015	11,286		325,932
New Hampshire				101,176	50,152	1.7				196,723
New Jersey							42,310	17,500	.1	655,273
New Mexico				312,863	190,813	16.7	222,461	135,695	14.0	480,264
New York	119,880	59,742	12.8	1,906,460	953,230	126.2	318,900	159,450	19.6	1,300,351
North Carolina	140,740	70,370	33.1	439,400	219,700	46.6	284,160	136,990	36.7	756,497
North Dakota										787,252
Ohio				184,400	92,200	3.8				1,739,441
Oklahoma							16,888	8,986		1,175,929
Oregon	114,890	62,115	33.8	24,833	15,177	1.5	92,641	55,525	13.7	695,340
Pennsylvania				91,266	45,633	5.9	936,177	456,869	65.4	1,642,716
Rhode Island	20,710	10,355	.3	118,988	59,494	3.6	89,590	44,795	2.6	132,231
South Carolina				52,688	24,000	8.2	454,850	191,200	39.0	463,480
South Dakota				11,300	6,250					816,436
Tennessee				152,246	76,123	7.2	42,680	21,340	.7	962,027
Texas				91,970	45,985	21.2	1,196,411	576,888	190.0	2,526,198
Utah	16,500	11,750	2.1	150,060	103,180	17.9	247,050	129,575	14.5	327,211
Vermont	77,950	38,975	4.3	192,650	84,150	11.6	3,750	1,875		121,875
Virginia	25,290	12,645	3.2	156,927	76,248	23.6	218,590	109,295	28.3	721,461
Washington	55,927	29,486	14.3	258,810	135,908	16.2	138,212	72,800	12.0	553,648
West Virginia				29,900	14,950	1.7	190,580	95,290	14.8	439,434
Wisconsin	30,710	15,250	6.7	60,026	27,300		230,842	103,700	8.2	1,081,228
Wyoming	75,809	46,849	7.2	255,110	157,630	23.9	123,310	76,190	19.9	351,373
District of Columbia										
Hawaii							54,010	27,005	2.4	219,870
Puerto Rico				217,720	108,860	11.2				138,015
TOTALS	2,076,754	1,071,901	476.2	8,371,597	4,298,474	656.9	8,626,533	4,142,031	877.8	39,862,594

* INCLUDES FUNDS APPORTIONED FOR THE FISCAL YEAR 1939

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF MARCH 31, 1938

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,933,286	$ 3,884,535	136.9	$ 260,300	$ 260,300	7.8				$ 6,280
Arizona	2,569,841	3,183,466	2,531,293	193.7	38,548	38,548					
Arkansas	3,352,061	3,163,813	3,133,779	351.9	165,042	164,942	8.6	$ 49,746	$ 49,746	4.0	3,594
California	7,747,928	7,876,172	7,496,923	263.6	217,622	217,579	.3				33,427
Colorado	3,395,263	2,382,800	2,301,910	99.4	97,797	95,414	6.0				997,939
Connecticut	1,418,709	1,378,662	1,263,206	21.2	55,490	55,000	1.2	124,130	64,435	.2	36,067
Delaware	900,310	871,469	843,920	66.4	10,234	10,234	.2	26,712	17,000	4.4	29,156
Florida	2,597,144	2,601,135	2,526,181	99.1	38,957	38,957					32,006
Georgia	4,988,967	1,948,280	1,894,278	112.3	2,363,094	2,094,714	112.5	581,320	581,320	30.2	418,655
Idaho	2,222,747	2,274,551	2,167,058	185.9	33,341	33,341					22,348
Illinois	8,694,009	8,124,376	7,849,991	465.2	685,010	685,010	23.2				159,008
Indiana	4,941,255	5,211,435	4,816,080	238.0	49,000	49,000					76,175
Iowa	4,991,664	5,239,568	4,885,935	528.3	105,902	104,865	.3				864
Kansas	4,994,975	4,728,537	4,674,834	376.2	250,210	248,037	16.5	8,800	8,800	.1	63,303
Kentucky	3,726,271	3,602,255	3,426,460	355.0	274,897	274,897	3.5				24,914
Louisiana	2,890,429	2,936,860	2,590,622	166.3	239,168	200,933	1.5	97,087	97,070	10.4	1,803
Maine	1,676,799	1,628,430	1,608,980	75.3	62,149	62,149	1.7				5,670
Maryland	1,750,738	773,435	766,648	27.2	467,536	467,536	10.0	233,764	220,812	4.4	295,742
Massachusetts	3,262,885	2,222,067	2,221,883	18.2	896,550	458,710	.6	1,042,586	521,293	.4	60,999
Michigan	6,301,414	6,658,140	5,949,537	288.6	284,921	284,921	3.3	223,938	63,746	.3	3,210
Minnesota	5,277,145	6,417,998	5,174,695	901.8	84,950	79,000					23,450
Mississippi	3,457,552	3,233,596	3,228,502	226.5	158,650	157,610	9.2	10,800	10,800	.6	60,641
Missouri	6,012,652	5,294,408	5,208,487	776.9	790,784	736,082	.7	34,391	32,294		35,789
Montana	3,676,416	3,597,647	3,566,706	200.7	95,385	95,385	.1	8,462	8,462		5,863
Nebraska	3,870,739	3,433,482	3,321,687	362.3	446,353	446,353	8.3	70,270	70,270	1.8	32,429
Nevada	2,243,074	2,314,381	2,197,401	110.1	84,970	38,146	1.7				7,527
New Hampshire	945,225	879,925	843,253	37.7	121,092	101,301	5.7				671
New Jersey	3,129,805	1,615,032	1,549,924	31.1	1,512,979	1,512,979	4.4	34,468	34,468		32,434
New Mexico	2,871,397	2,811,688	2,805,816	213.7	43,071	43,071		14,681	12,196		10,314
New York	11,046,377	10,709,139	10,243,718	170.0	316,300	276,300	1.9	70,163	70,163	.2	456,196
North Carolina	4,720,173	4,614,368	4,543,928	295.4	120,761	120,761	5.4	17,090	9,816		45,669
North Dakota	2,867,245	2,488,427	2,440,492	377.9	107,799	107,799	1.2	293,869	293,869	37.5	25,085
Ohio	7,670,815	6,977,466	6,861,887	294.9	703,332	697,332	3.7				111,596
Oklahoma	4,580,670	4,563,146	4,318,793	400.8	239,470	239,470	7.6	8,800	5,000	.1	17,407
Oregon	3,038,642	3,197,374	2,970,434	164.4	45,580	45,580		11,846	11,846	1.0	10,783
Pennsylvania	9,347,797	8,022,311	7,438,538	262.7	1,999,961	1,743,728	20.0	10,356	10,356	.2	155,175
Rhode Island	989,208	1,113,140	989,208	18.8							
South Carolina	2,702,012	2,356,294	2,150,784	226.2	551,692	498,752	23.4	9,664	9,664	1.5	42,811
South Dakota	2,976,454	2,681,291	2,676,644	482.9	270,437	270,437	22.0				29,373
Tennessee	4,192,460	3,529,833	3,472,059	135.3	713,268	713,268	18.0				7,133
Texas	11,989,350	12,693,455	11,567,575	1,112.8	415,219	377,018	11.9				44,758
Utah	2,067,154	2,161,834	1,925,059	207.7	112,055	112,055	.1				30,040
Vermont	924,306	1,057,889	902,441	23.2	13,865	13,865					8,000
Virginia	3,652,667	3,475,607	3,285,509	939.9	131,731	128,434	13.5	156,000	156,000	1.1	82,724
Washington	3,026,161	3,364,343	2,940,420	164.2	65,824	65,824	.7				19,916
West Virginia	2,231,412	2,109,240	1,963,718	86.4	275,023	267,639	14.1				55
Wisconsin	4,823,884	5,217,220	4,727,229	343.4	91,394	89,300	.3				7,355
Wyoming	2,219,155	2,173,387	2,165,009	152.4	33,287	33,287					20,859
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	955,853	871,389	17.4				62,530	54,644	.6	
TOTALS	195,000,000	184,748,511	174,134,854	12,815.0	16,141,000	14,855,863	371.1	3,201,473	2,414,070	99.0	3,595,213

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF MARCH 31, 1938

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
				NUMBER					NUMBER					NUMBER			
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,599,166	$ 3,583,815	48	1	12	$ 503,319	$ 437,519	5								$ 13,283
Arizona	1,256,099	1,278,187	1,216,371	15		5	18,841	18,841									20,887
Arkansas	3,574,060	2,922,486	2,914,937	51	6	31	611,406	609,674	5		2	$ 41,806	$ 41,806	1			7,643
California	7,4[illegible]6,362	7,359,088	7,189,833	47	8		112,295	112,295				10,000	10,000	5			174,234
Colorado	2,631,567	2,319,850	2,249,587	27	3	21	355,944	355,944	3								26,036
Connecticut	1,712,684	479,557	479,463	3	1	1	1,157,071	1,135,710	7								97,511
Delaware	418,239	130,000	128,144	1			279,052	279,052	2								11,043
Florida	2,827,883	2,396,864	2,377,173	30	5	3	119,018	119,018	1			202,300	202,300	1		73	129,391
Georgia	4,8[illegible]5,949	799,536	790,230	17	7	19	1,505,839	1,505,839	24	7		910,250	910,250	10	2	38	1,689,630
Idaho	1,674,479	1,393,559	1,365,205	20	3	12	252,696	252,696	2		6	4,261	4,261			3	52,317
Illinois	10,307,184	8,786,476	8,659,771	65	8		1,601,134	1,594,730	11			47,000	18,000	2			34,683
Indiana	5,111,096	4,774,115	4,632,083	42	15		450,720	450,720			162						28,293
Iowa	5,600,679	4,568,180	4,469,381	98	9	8	1,092,875	1,089,000	8								41,798
Kansas	5,246,258	4,410,068	3,984,232	55		5	1,197,632	1,178,217	3	1	2	112,152	19,148	2		1	64,661
Kentucky	3,672,387	1,358,467	1,339,494	19	5		1,984,219	1,694,487	5	6		612,579	612,579	3	1		25,827
Louisiana	3,213,467	1,767,076	1,758,112	18	2		782,183	782,153	8			644,917	595,438	4	1		77,763
Maine	1,426,861	1,267,924	1,263,008	19	1	4	160,788	142,540	1	1							21,313
Maryland	2,061,751	701,589	696,981	5	4	18	844,674	794,674	5	1	11	283,163	282,277	1	1	1	287,819
Massachusetts	4,210,833	2,993,173	2,990,739	22	4		865,680	865,680	4			249,991	249,991	1			104,423
Michigan	6,765,197	6,864,894	6,560,765	44	8		101,477	101,477				167,000	75,730	1	1		27,225
Minnesota	5,395,441	4,824,289	4,787,124	83	13	50	590,070	584,020	3								24,297
Mississippi	3,241,475	2,572,055	2,568,447	53	6	14	192,800	192,800	4	1	1	317,200	317,200	4		8	163,028
Missouri	6,142,153	4,717,600	4,552,409	42	1		1,566,960	1,566,960	7		1	1,650	1,650			1	21,134
Montana	2,722,327	2,551,613	2,507,468	37	7		245,576	185,931	1	1							28,928
Nebraska	3,556,441	2,889,896	2,840,725	76	3	22	472,542	472,542	5		2	194,329	194,329	3			48,845
Nevada	887,260	877,374	864,025	8	3		13,308	13,308				3,630	3,630			5	6,297
New Hampshire	822,484	791,275	791,208	9	4	6	53,297	29,170	1			2,106	2,106			1	
New Jersey	3,983,826	3,107,456	3,094,275	20	6	1	772,334	772,334	4	3		84,730	84,730		1		32,487
New Mexico	1,725,286	1,700,885	1,693,233	19	1		25,879	25,879									6,174
New York	13,577,189	11,691,115	11,348,097	35	47		1,877,450	1,877,450	10	2		155,700	155,550		1		196,092
North Carolina	4,823,958	3,615,380	3,595,570	50	18		1,081,059	1,081,059	12		115	118,900	118,900	2			28,429
North Dakota	3,207,473	2,803,739	2,795,735	55	4		389,012	389,012		1							22,726
Ohio	8,439,397	2,476,397	2,355,149	19	6	13	5,653,537	5,185,391	30	2		670,000	655,000	10		1	244,357
Oklahoma	5,004,711	3,938,622	3,921,634	60	9	40	1,141,944	1,008,844	5	3		1,350	1,350			1	72,883
Oregon	2,334,204	2,294,033	2,249,653	16	6	2	74,215	74,215	1								10,336
Pennsylvania	11,483,513	9,075,239	8,362,371	71	18	9	2,850,615	2,683,567	15	5		150,473	150,473	5			287,202
Rhode Island	699,691	701,817	692,910	4	3												6,781
South Carolina	3,059,956	2,118,895	2,093,187	40	12	35	515,927	515,927	5	4	9	134,706	134,706	1	1	4	316,136
South Dakota	3,249,086	2,832,595	2,830,936	60	5	53	175,186	175,186	6			269,350	210,392	17			32,572
Tennessee	3,903,979	2,638,938	2,619,360	40	3	38	1,037,450	1,037,450	6	1	2						247,169
Texas	10,855,982	9,968,860	9,955,957	127	14	117	476,050	476,050	1	1	13	652,760	336,280	1		19	87,696
Utah	1,230,763	1,203,470	1,193,602	17	1	1	18,461	18,461									18,700
Vermont	729,857	752,414	706,303	10	7	20	10,900	10,900									12,654
Virginia	3,774,287	3,008,095	2,870,531	45	19	23	848,268	841,643	7	2		4,506	4,506			1	57,607
Washington	3,095,041	2,987,234	2,964,050	22	12	11	91,167	89,782	1								41,210
West Virginia	2,677,937	1,559,891	1,557,923	14	3	6	1,037,711	1,037,211	12			78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,481,639	4,437,446	37	6	10	399,472	399,269		1	10	105,662	51,212	1		14	134,756
Wyoming	1,360,841	1,213,957	1,203,739	13			111,212	111,212	1								45,890
District of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			179,710	169,698	2								
TOTALS	196,000,000	154,267,698	150,797,700	1734	327	610	35,899,475	34,545,537	233	43	336	6,231,326	5,522,649	77	9	172	5,134,114

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 3 MAY 1938

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 3 May 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

AN ACTION PROGRAM TO ADVANCE SAFETY ON THE HIGHWAYS[1]

By THOS. H. MacDONALD, Chief, United States Bureau of Public Roads

THE ADVANCEMENT of safety on the streets and highways of the nation requires definite action on many fronts. The desirable objective for the year 1938 is a vigorous, concerted movement to bring into being on an effective scale the measures and methods which the present knowledge of the character and scope of the problems sustains as a sound program. Perhaps the most important single principle developed during the year just closed of extensive study by the Bureau of Public Roads and the Highway Research Board of many phases of this subject is that there is no single cause of accidents. Each accident is the result of causes in combination. There are always present more than a single element and the number of these elements that can and do contribute to accidents is very large.

The possible number of combinations of these elements by mathematical calculation produces a potentiality of accidents of a magnitude beyond conception. Probably the failure of the lawmakers to recognize this characteristic of multiple contributing elements in each accident has acted as a powerful deterrent to more effective programs of accident control. Certainly it has given rise to serious delays in formulating and applying sound preventive measures. For example, the hope that a single panacea exists that will stop accidents, although utterly absurd, results in all sorts of cure-all offerings, many of them from so-called experts or near experts.

Since there is no single cause there can be no single panacea for accidents. On the other hand, the recognition by the courts of multiple accident-contributing elements has resulted adversely to the imposition of even reasonable penalties upon offenders of the traffic laws. Here the recognition of the contributing factors mitigates to the point of allowing, in many cases, the offender to escape without price the most serious infractions of the law. So widespread is this experience as to lead to the reasonable conclusion that reliance for any great assistance in accident prevention does not lie in the courts as they are now operated in the field of traffic regulation.

Sufficient recognition has not as yet been given to the fact that this big problem of street and highway safety throws upon the established routine and traditions of the courts a new social problem of tremendous dimensions with which they are not designed to cope. The processes of the courts are not singled out for criticism in themselves but to illustrate the failure of society as a whole to recognize the tremendous dimensions of the street and highway traffic safety problems and to provide adequate methods and organizations to deal with these problems effectively.

More than a year ago the Congress directed the Bureau of Public Roads to undertake a study of the problem involved and to submit a report. Acting in cooperation with the Highway Research Board the Bureau has produced a report, or rather a series of reports, dealing with the following subjects:

1. Nonuniformity of State motor-vehicle traffic laws.
2. Skilled investigation at the scene of the accident needed to develop causes.
3. Inadequacy of State motor-vehicle accident reporting.
4. Official inspection of vehicles.
5. Case histories of fatal highway accidents.
6. The accident-prone driver.

These reports have all been transmitted to Congress and are being printed as House Document No. 462. By acting through the Highway Research Board not only the previous studies which had been made by that Board were available, but it was possible to secure the assistance of the individuals who were already working in the several fields of investigation and who have devoted their best efforts to the studies and the resulting reports. Further, a number of the leading authorities, representative of organizations primarily concerned in the traffic safety field, served as an advisory committee to make suggestions as to the scope of the studies and the production of the report.

It is not the purpose of this paper to review the findings of these reports, but, on the basis of the material developed, to propose an action program for the immediate future and to fix in so far as possible the responsibility for the initiation of this program. Neither is it the purpose to prove by the use of statistical material the urgent necessity for enlarging, and quickening the tempo of, sound national programs directed toward the control of the tragic record of accidents and fatalities. A responsible attitude of good citizenship and intelligent selfishness places the road builders and road building industry of the nation behind every effort to advance the cause of highway safety.

In this social problem already grown to such alarming dimensions that it ranks as one of our greatest national problems, while most of the implications and indications are discouraging, there is one important encouragement. Under the coordinating influence of the Automotive Safety Foundation, 12 national organizations which are most intimately and directly concerned with traffic safety problems have united in accepting and recommending a comprehensive model program for each State to increase traffic efficiency and to reduce accidents. This program brings into a coordinate entity the elements of legislation, motor-vehicle administration, enforcement, engineering, education, training personnel, and research. It places responsibility for action upon the governor, the State officials, and the legislature supported by a nonofficial organization of civic and business groups. The combination of official and public groups is conceived to constitute the State safety organization. With this program the Bureau of Public Roads is in entire accord and holds it to be indicated by all the important studies and researches which have been made in this field.

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[1] Paper presented at the Thirty-fifth Annual Convention of the American Road Builders' Association, Cleveland, Ohio, January 17, 1938.

While it may appear to the public at large that inadequate progress is being made in advancing the cause of highway safety, it certainly holds promise of a greatly accelerated rate of progress that all the most important organizations engaged in this field have agreed upon the program most needed and have joined their efforts toward the common end of making the program effective upon a national scale. The effort of this paper therefore is to reduce to concrete terms the necessary steps to be taken and in so far as possible to suggest the responsibility of initiation of the component parts of the program. Only by action on a national scale can even reasonable progress be made in the reduction of accidents and in the freeing of our streets and highways from hazards to permit the enjoyment of their convenience and service.

To carry into effect the existing knowledge in the field of accident prevention, the following eight-point program is submitted. It is not intended to include all of the steps that will be necessary, but it does provide a sound foundation for traffic control and regulation in the interests of traffic safety. Neither are the elements of this program independent but on the contrary they are inter-dependent, and to an extent overlapping. Several of these which are important parts of others are given individuality for the purpose of emphasis. The action program recommended follows:

1. Uniform State motor-vehicle traffic laws.
2. Skilled investigation of traffic accidents.
3. The establishment of a uniform system of accident reporting.
4. The establishment of an adequate highway patrol including the official inspection of vehicles.
5. The establishment of complete and final authority over the issuance and revocation of drivers' licenses.
6. A highway improvement program divided into two general classes of projects:
   - *a*. Those of the emergency type.
   - *b*. Those for the long-time plan.
7. A plan of State and Federal safety organization adequate to secure on a wide scale the adoption and enforcement of the action program here proposed.
8. A national educational program.

The scope of each element of this program is indicated but not intended to be fully developed in this discussion.

1. *Uniform State motor-vehicle traffic laws.*—This subject has received long and careful attention. Through the organized effort of the National Conference on Street and Highway Safety a series of five proposed model laws were drafted and have been widely distributed. While a considerable number of States have adopted these laws in whole or in part there yet remains a serious nonuniformity in the major principles which affect highway safety. It is not proposed that the motor-vehicle laws must be identical in every detail, but they must be in those matters which even remotely affect the safety of motor-vehicle operation. The driving habits of individuals tend to become fixed, and particularly in times of stress or emergency the reactions are involuntary. Now that the traffic between communities and between States has become universal the necessity for replacing uncertainty of the law with certainty is self apparent. The necessity for drivers knowing that they do not face new traffic laws or conditions when they cross State borders cannot be denied. But the existing condition is much more serious than this as indicated in these examples:

> Eight States do not require that all motor-vehicle operators be licensed.
>
> Two States fail to provide a minimum age limit for the driving of a motor vehicle.
>
> Fourteen States permit the operation of motor vehicles by minors 14 or 15 years old.
>
> Twenty-two States do not prohibit the issuance of drivers' licenses to habitual drunkards or narcotic addicts.
>
> Twenty-four States do not prohibit the issuance of drivers' licenses to those afflicted with mental disabilities.
>
> Only 11 States require drivers to be able to read highway warning signs printed in English.
>
> In 6 States which do require the licensing of motor-vehicle operators, no examination or test is included as a prerequisite to the issuance of such license, and in 5 others the applicants may be examined only when the official issuing the license suspects they are unfit to drive.
>
> Only 30 States require every applicant for a driver's license to be examined for competency and in these the examination varies widely.
>
> Only 8 States require a test of vision, 15 a knowledge of traffic laws.

Unquestionably there should be a strict uniform license law permitting licenses to be issued only after a competent examination including tests of vision, ability to read traffic signs, knowledge of traffic laws, and demonstration of ability to operate the vehicle.

In only 27 States has there been adopted a manual of uniform traffic control devices. The rules of the road governing the operation of vehicles upon the highway vary so greatly from State to State that confusion is inevitable. Nine States do not require vehicles to be driven to the right of the highway at all times. Six States do not provide that passing must be on the left of the overtaken vehicle. Passing on curves is specifically prohibited in 36 States and on the crest of hills in 35 States, at intersections with other highways in 34 States. The provisions with respect to the equipment of motor vehicles and physical characteristics such as length, width, and height, have a wide variation with many inconsistencies.

Further discussion of this point is hardly necessary to indicate the fundamental necessity of a uniform motor-vehicle law between the States in the interests of safety. The only possibility of providing for this part of the action program lies in the adoption by the State legislatures of the necessary uniform laws. A guide to the points of divergence from the uniform code will be found in the bulletin Nonuniformity of State Motor-Vehicle Traffic Laws that is available for distribution by the Bureau of Public Roads.

2. *Skilled investigation of traffic accidents.*—An intelligent attack on the highway accident situation must depend upon two details which, while simple in concept, are not easy of achievement. There must be information as to the circumstances under which each accident occurs, and there must be sufficient analysis to record where, and the manner in which, accidents are happening repeatedly. Accident data are essential to an intelligent and efficient police enforcement program within the urban districts, and the highway patrol can provide important information in the rural districts. Within the urban areas the establishment of especially trained squads which are ready on call to investigate each accident without loss of time is essential. It may not be practicable at once to establish accident squads in the rural police organization sufficiently extensive to investigate all accidents, but it is possible to establish a number of special accident squads and to train each member of the highway patrol in the correct procedure to investigate and to report accidents so that a high general level of accuracy in reporting may be attained.

So long as we depend for accident data on ex parte information and make inadequate provision for the securing of reliable and disinterested reports, just so long will our knowledge of the facts be insufficient and open to question. It will not be possible to secure impressive accident reduction until it is recognized that every accident is the resultant of a number of causes and circumstances in the absence of any one of which the accident would not have occurred or would have had different consequences. No contributing cause is so unimportant that it may be overlooked or dismissed. That only a beginning has been made in this field in a very few States is convincingly revealed by the analysis presented in the case histories of the fatal highway accidents published by the Bureau of Public Roads. Certainly the establishment of a complete system of accident investigation and reporting will require much more adequate organization than now exists but the cost will be small in comparison with the losses in life and property.

3. *The establishment of a uniform system of accident reporting.*—The uniform reporting of accidents is partially covered by the preceding discussion, but requires in addition the adoption of a national standard form. Much work has been done upon the formulation of such a form by the National Safety Council, the Bureau of the Census, and others, and further work is now in progress. It is essential that the terms be clearly defined, and the statistical methods employed give accurate results. In the studies made by the Bureau of Public Roads and the Highway Research Board the lack of accurate and sufficient statistical data relating to accidents, even those involving fatalities, was an insuperable obstacle in the production of a more determinate report. For example, the national reports of motor-vehicle fatalities are divided between urban and rural on the basis that urban accidents are those in cities of 10,000 and upwards, and all others are classified as rural. Taking into consideration the fact that a very large percent of the total population lives in places of, say, 1,500 to 10,000 people, it is inevitable that a large percentage of fatalities reported in the rural category actually involve urban and not rural conditions. This probable discrepancy is further accented by the fact that so large a percentage of fatal accidents involve pedestrians. It is apparent that effective efforts to combat accidents are utterly dependent upon accurate and complete statistical data.

4. *The establishment of an adequate highway patrol including the official inspection of vehicles.*—In most of the States the highway patrol organization for the rural districts, and in most of the cities, the traffic police, are wholly inadequate in numbers and equipment to patrol the mileage over which there is appreciable traffic. This condition of inadequacy in numbers is further aggravated by the addition of so many miscellaneous duties that too little time is left for patrol work. The most complete code of motor-vehicle laws is of little avail unless there is proper enforcement. An increase in the number of trained patrolmen in every State is necessary.

Only 23 States now provide for the periodical inspection of motor vehicles, but all of these do not include all types. The inspection laws vary in detail but are uniform in the attempt to prevent the operation of mechanically defective vehicles. The establishment of inspection requirements for the safe mechanical condition of motor vehicles and their continuous enforcement is an important "must" in the action program.

5. *The establishment of complete and final authority over the issuance and revocation of drivers' licenses.*—In the development of this discussion the importance of the official charged with the administration of the motor-vehicle laws must be more and more apparent. The lack of essential laws in a large number of States fixing the qualifications of those who are permitted to operate motor vehicles upon the public ways has been recorded. The disclosures in the bulletin upon the accident-prone driver sustain the conclusion that the motor-vehicle administrator must have authority to suspend or revoke the license of operators, and that this power must be exercised. The discretionary authority to revoke licenses should by law be obligatory action in the more serious offenses. Thirty States now require that a license be revoked for conviction of manslaughter; 36 States, for driving while under the influence of intoxicants; 31 States, for failure to stop after an accident; and 25 States, for reckless driving. Certainly these offenses justify the revocation of licenses in each State.

But the fact of the existence of the accident-prone driver, which is now established beyond reasonable doubt, justifies the suspending or revoking of licenses for much less grave offenses than these enumerated. Twenty-three States have recognized this by providing discretion to suspend or revoke for habitual recklessness or negligence, 22 States provide the same discretionary authority to revoke the license of drivers involved in a serious accident, and 11 States for any sufficient cause. The recognition of the responsibility resting upon the individual driver for accident prevention so far outweighs any other consideration that a sound action program must include, as the most effective weapon of society in attacking the problem of accident prevention, the generally exercised power to revoke drivers' licenses for infraction of the traffic laws.

6. *A highway improvement program divided into two general classes of projects: (a) Those of the emergency type, and (b) those for the long-time plan.*—This year the Statewide transport surveys carried on in cooperation between the State highway departments and the Bureau of Public Roads will begin to reach usable form. There will be available for the first time in any State complete information as to the types and conditions of highways, the amount and character of use, and all other information which can conceivably have a direct relationship to the safe use of the highways.

The extent to which highway conditions enter as a contributing factor into more serious accidents is unknown and to an extent indeterminate. Much that has been said in criticism of highway conditions confuses the fundamental issues of highway safety and highway capacity measured by speed and number of units. Between 1925 and 1936 highway usage measured in terms of gasoline consumption increased by an estimated amount of 110 percent, while registration in numbers of units increased 42 percent. The gas consumption per registered vehicle has increased just under 50 percent. From these facts it is apparent that motor vehicles are being used on a steadily increasing rate per average unit, and even with these increases the rate of accidents measured in miles of vehicle operation shows a decrease.

With the definite facts of the public use of highways in every community of the nation, it is readily apparent that the providing of safer highways must be through the elimination over large mileages of those conditions which to the best of our knowledge are most likely to be contributory factors in accidents. This is the keynote to the emergency program which should be put into effect determined by the data of the transport surveys. As an illustration of the type of work included—a rechecking of the system of warning signs and signals, the relocating of these to fit the generally increased speed, the elimination of unnecessary signs, the marking of all curves with safe driving speeds if below the legal limits, the correction of sight distances particularly at intersecting roads, and the widening and maintenance of shoulders.

The increased operating speeds demand wider roads and the narrower widths which were reasonably safe for speeds of a few years ago must be supplemented by nonskid material, the shoulders widened, slopes flattened, and deep ditches eliminated. For the long-time program, particularly for new multiple-lane roads, all the accepted elements of safe design to permit continuous safe traffic flow, including the division of opposing lanes by a neutral area and the elimination of cross traffic, should be provided. Separation of grades between the railroads and important intersecting highways must be a first consideration.

7. *A plan of State and Federal safety organization adequate to secure on a wide scale the adoption and enforcement of the action program here proposed.*—Just how the Federal Government fits into the general scheme of an adequate traffic safety organization is not clear beyond debate, but there are considerations of public policy which prompt the suggestion that no wholly new board, commission, or other Federal authority is desirable. To be effective, such a Federal organization would have to duplicate organization, authority, and operations now lodged in some Federal department. There is need for a correlation of the Federal activities and relationships between the Federal departments themselves, and between the Federal Government and the States in both the official and public aspects. A plan of Federal organization that can be made effective at once and make use of the going operations of the

(Continued on page 51)

AN AUTOMATIC RECORDER FOR COUNTING HIGHWAY TRAFFIC

Reported by R. E. CRAIG, Junior Highway Engineer, Division of Highway Transport, United States Bureau of Public Roads

IN PREPARING the initial plans for the State-wide highway planning surveys now being conducted by the highway departments of 43 States in cooperation with the Bureau of Public Roads, it was realized that, in order to obtain anything like complete traffic data for the entire rural mileage, counts on a large part of this system must be made on a limited schedule; and it was believed that continuous records showing typical traffic patterns would be invaluable in estimating traffic from these short counts. In addition, these records were needed to provide a basis for analyzing various daily, weekly, seasonal, and other traffic patterns in determining the relative reliability of various methods of sampling traffic. The need of a better basis than had previously existed for predicting future traffic was still another reason for obtaining continuous records. However, the cost of obtaining such records by manual counts was so great that their accumulation, except in a few experimental operations, could not be justified by the value of the results.

Before the first State-wide highway planning surveys were begun late in 1935, sporadic efforts had already been made to develop a counting and recording machine which would supply traffic records economically. The immediate need for this equipment in conducting highway planning surveys lent sufficient impetus to these efforts to make possible the construction of a satisfactory model in the fall of 1935.

The real problem in constructing a satisfactory automatic traffic recorder was the development of a reliable means of detecting the passage of vehicles, preferably some method which would not involve the use of a detector pad laid on the surface or inset in the pavement, and which could be produced and operated at reasonable cost.

After investigation of available equipment and detailed consideration of a number of suggested methods it was concluded that the most practical method for detecting the passage of a vehicle at permanent installations was the adaptation of the photoelectric cell. This cell is based on the principle that the conductivity of certain elements is proportional to the amount of light falling on the element. Photoelectric equipment for counting purposes had been in use for a sufficient length of time to demonstrate its efficiency and reliability under controlled conditions.

The recorder developed was designed to count passing vehicles without counting pedestrians. This was done by using two parallel beams of light, approximately 30 inches apart, directed across the roadway upon photoelectric cells. A standard photoelectric relay unit is designed as a switch to be operated by the simultaneous interruption of both light beams. This switch (the counter relay) operates the contact which, in turn, controls and operates the counting circuit. Pedestrians normally interrupt only one beam at a time and are not recorded.

The photoelectric tubes are very sensitive and can be adjusted to record vehicles passing at high speeds. The machine is designed to count as many as 24,000 interruptions of the light beams per hour. Infrared ray filters were installed in the light source to render the beams practically invisible, thus eliminating hazard and annoyance to passing drivers. Figures 1 and 2 show one of the types of photoelectric automatic traffic recorders being used in the highway planning surveys. Two types of automatic recorders are shown in figure 3.

The recording mechanism is designed to print on standard adding machine tape once every hour, recording the day, hour, and minute, and the cumulative traffic total. Figure 4 shows a sample section of tape. The number of vehicles passing each hour is obtained by subtracting each total from the succeeding total. Postmeridian time is indicated by a dash under the hour. The recorder is timed by a self-starting synchronous motor of the type used in electric clocks.

The recording or printing circuit is designed so that if the current fails, the recorder will stamp automatically the time of the failure and the cumulative counter reading. In case the recorder clock is found to be slow upon inspection, the time at which the power failure occurred is recorded and the proper corrections can be made to the record if there has been only one interruption of current.

INSTALLATION OF TRAFFIC RECORDER DESCRIBED

Manufacturers can supply machines with auxiliary mechanical clocks to keep the equipment on time in the event of power failures. However, the dependability of the power supply even on rural installations has proved to be such that the additional cost of this equipment has rarely been considered justified.

The recorders now being used count and record the total number of vehicles passing the point at which they are installed. However, they can easily be adapted to count and record only vehicles proceeding in one direction. This can be accomplished by adding relays that will permit electrical impulses to be sent to the counting mechanism only when the beams of light are interrupted in the proper order.

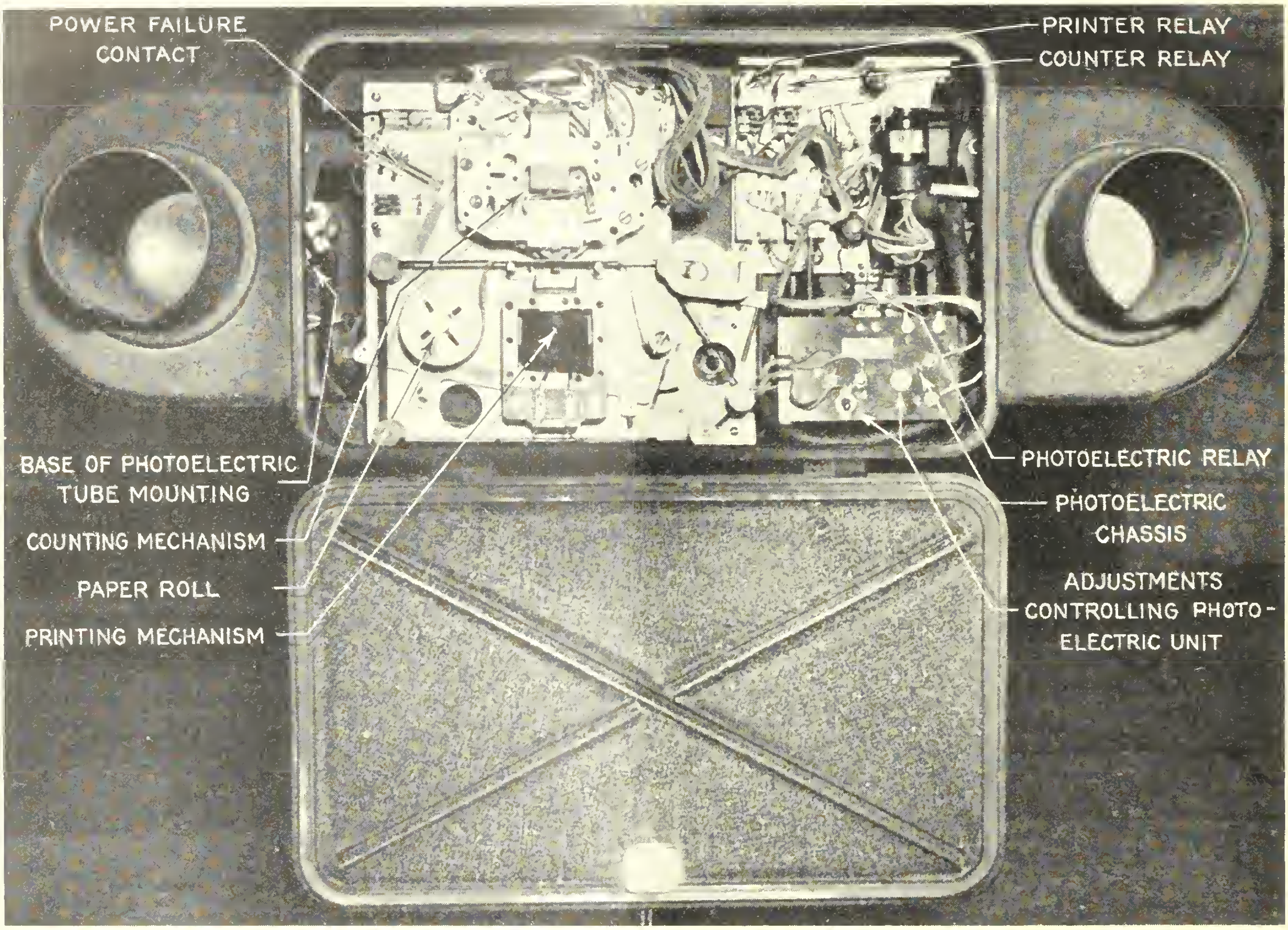


Figure 1.—Recording and Printing Apparatus in One Type of Automatic Traffic Recorder.

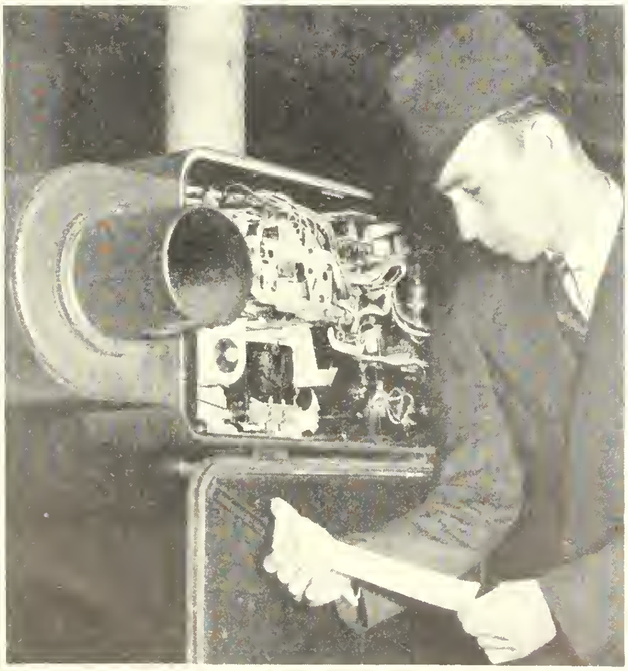


Figure 2.—Inspecting a Record Tape Removed From a Recorder.

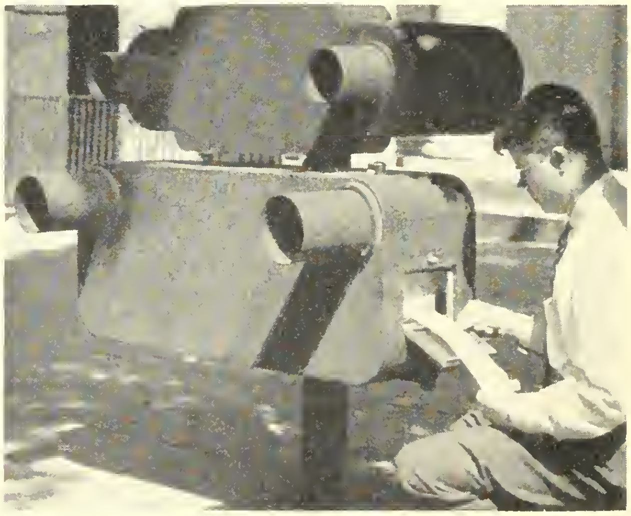


Figure 3.—Two Types of Automatic Traffic Recorders.

In the early experimental work, an effort was made to count trucks and busses independently of the total traffic by placing a third light beam, operating an independent counter, at a sufficient height above the roadway to clear the tops of passenger cars. Satisfactory results were not obtained with this unit, however, because a large number of light trucks are approximately the same height as passenger cars and were not recorded; in addition, an appreciable number of passing trucks were found to be so loaded that each registered two or three times. The count obtained with this device was not accurate and the third beam was not incorporated in the final design of the recorder. No serious effort has since been made to develop a traffic counter that will record trucks and busses independently, as sufficiently accurate estimates of the percentage of truck and bus traffic are ordinarily obtainable by making short manual counts.

The traffic counter operates on single-phase, 110-volt, 60-cycle alternating current and, therefore, is usually installed near electric transmission lines supplying such power and where there is little likelihood of current failure. Installations are usually made away from built-up sections, especially schoolhouses, where there is likely to be a grouping of pedestrians. Groups of pedestrians will frequently interrupt both light beams simultaneously, resulting in extra counts. The number of under-counts, resulting when two vehicles pass simultaneously in front of the recorder and are recorded as one vehicle, is reduced by selecting a location where there is little chance of vehicles traveling in the same direction passing each other, as at a narrow section of the road, on the top of a hill, or on a curve. Installations on 3-lane roads have, as a rule, been made only at points at which vehicles proceeding in the same direction are prohibited from passing. Satisfactory results on 4-lane roads have been obtained where the opposing lanes of traffic are divided and the recorders are installed to count traffic moving in one direction only.

Figure 5 illustrates a typical installation for recorders in ordinary locations. The light source and recording unit are mounted so that the light beams make an angle of approximately 23° with a line transverse to the roadway. By installing the equipment at a skew with the roadway, the projection of the light beam on a vehicle is such that it will not permit the beam to pass between the body of the vehicle and protruding equipment, such as trunks or spare tires. This eliminates a number of extra counts that would be caused by the interruption of the light beams by accessories if the beams were directed perpendicular to the centerline of the roadway. Experience has demonstrated that the proper height of the light beams above the roadway is approximately 3 feet 8 inches. At this height the beams do not pass through the windows of passenger cars, nor do they pass under trucks of ordinary design. When installed on curves the beams should be parallel to the superelevated surface. The specifications under which recorders have been purchased require that the equipment operate satisfactorily with a distance between the light source and receiving unit of 50 feet, and this is usually the maximum distance for field installations. However, the equipment has operated satisfactorily at distances considerably in excess of 50 feet.

The recorder and light source are mounted on posts consisting of 4-inch pipe set in concrete to a depth of 3 feet below the surface of the ground. The concrete is cast around a mold slightly larger than the pipe's outside diameter, and after the concrete has set, the pipe is slipped into the hole and properly secured. Wedges are used, if necessary, to assure rigidity. In some instances, pipe standards having threaded joints at the ground level have been used. The advantage of these methods is that the equipment can be easily moved to a new location when desirable.

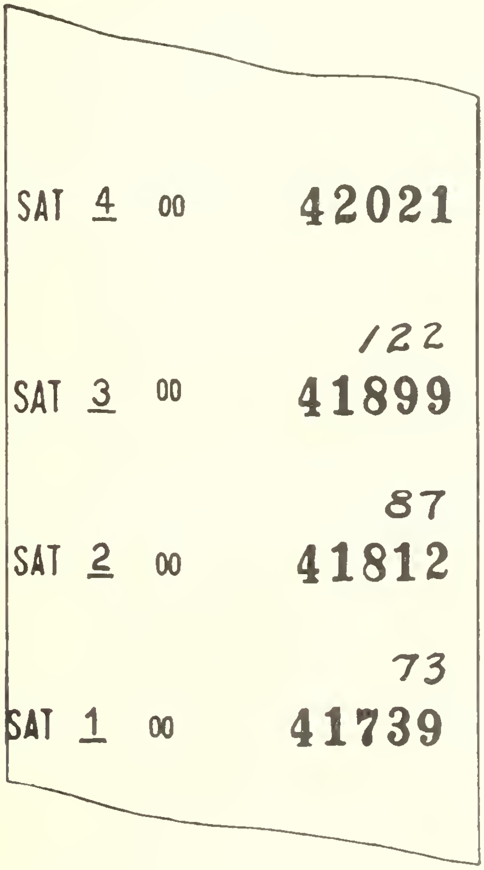
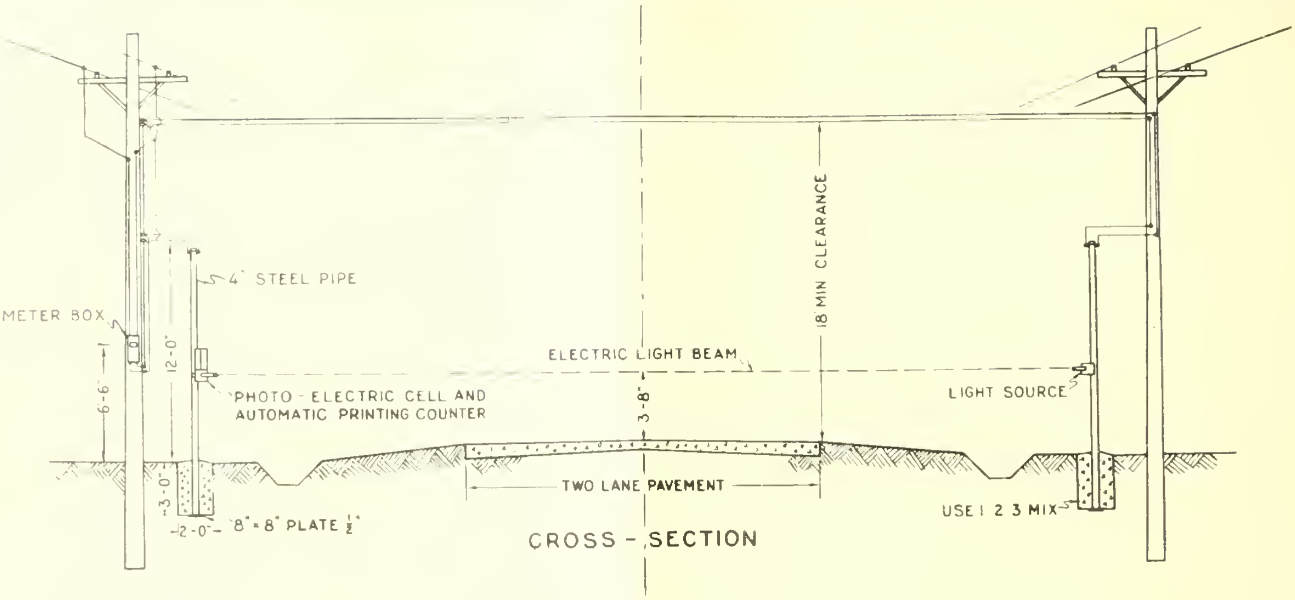
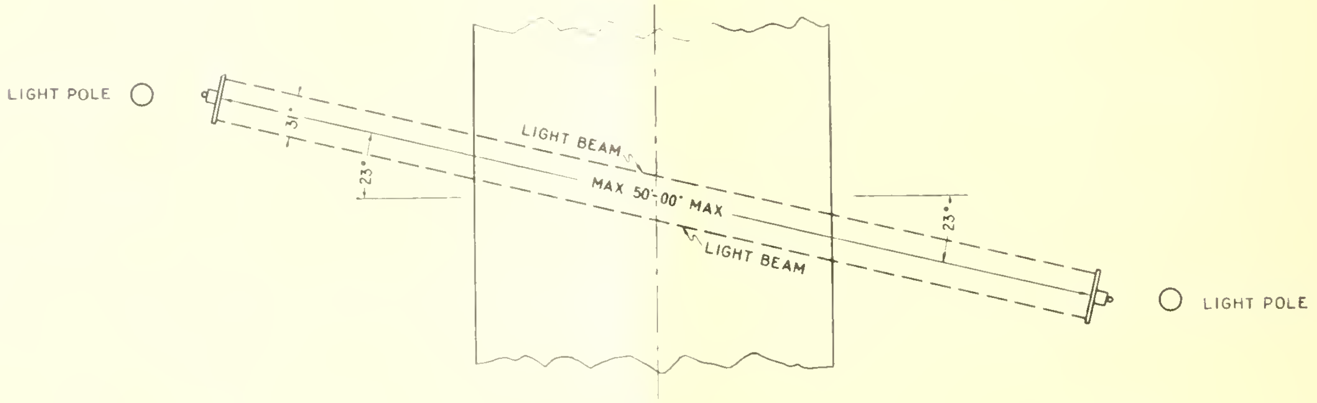


Figure 4.—Section of Record Tape From an Automatic Traffic Recorder.

A box containing a meter, fuses, and master switch, is attached to the power line pole. Connections are made from the master switch to the equipment, wires being run inside the pipe supports wherever possible. If the cross connections are carried over the roadway, care is taken to place them a safe height above the road. In installing cross connections, advantage has been taken of any ready means of carrying the connections under the roadway, such as through a culvert or underneath a bridge. All exposed wires are carried in waterproof conduits. Figure 6 shows a typical automatic recorder installation.



CROSS - SECTION



PLAN

Figure 5.—Typical Arrangement of Equipment in Installing Automatic Traffic Recorders.

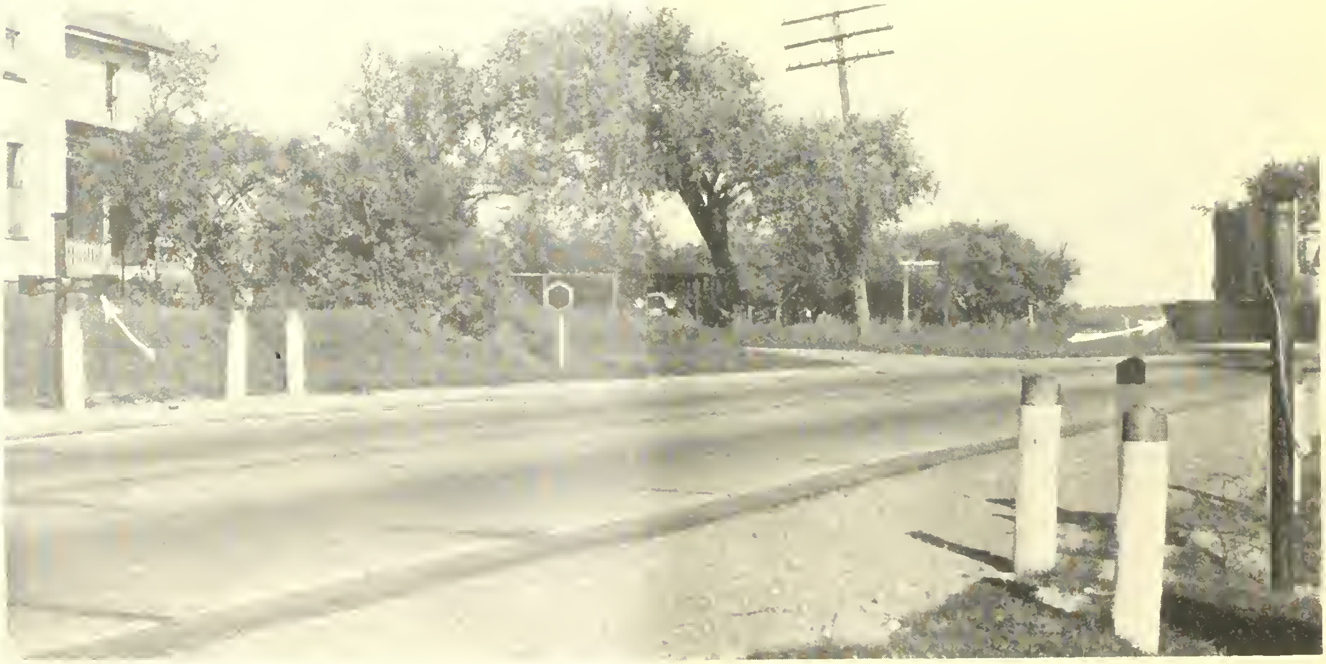


Figure 6.—Automatic Traffic Recorder Installation in Michigan. Note Guard Posts to Protect Recorder From Traffic.

Experience indicates that it is advisable to protect automatic recorders by installing them behind existing guard rails or natural barriers. Otherwise, it has been found necessary to install special guard rails to prevent damage to them.

MANY DIFFICULTIES OVERCOME IN OBTAINING SATISFACTORY PERFORMANCE

At the time the majority of the automatic recorders in use in the planning surveys were installed, knowledge of the performance of photoelectric equipment was limited largely to those who had done development work with them. Although the equipment was known to be affected by stray light and the intensity of the light beams directed on the photoelectric cells, many unanticipated difficulties were encountered.

One of the first problems was the abnormally short life of lamp bulbs in the light sources of several installations. For no apparent reason the lamp bulbs in these installations failed to give a reasonable length of service. Investigation led to the discovery that the voltage being supplied was in some instances as much as 20 or 30 volts above the required 110 volts. Untimely lamp failures were eliminated either by requiring the power company to supply the proper voltage or by installing resistance coils in the light source to reduce the voltage supplied to the lamps.

At different periods through the day machines would sometimes fail to count traffic or would record a large number of vehicles when only a few had passed the recorder. Apparently the trouble was not caused by stray light as a number of these failures were occurring at night. The trouble was corrected when methods of focusing the light beams were improved and the size of lamp bulbs used in the lamp source was increased from 32 to 50 candlepower.

In instances where insufficient light reached the elements because of condensation of moisture on the inside of the lenses in the receiving unit, heating elements were installed or a deliquescent substance was placed back of the lens to absorb moisture. These measures, however, did not completely solve all of the problems, as some recorders still performed erratically. A study of the conditions under which the recorders were operating revealed that intermittent operation was now confined largely to machines operating under weather conditions that produced high humidity. Moisture was collecting around the base of the photoelectric tubes, causing a leakage of current that resulted in erratic operation of the counter. To eliminate this leakage, heaters were installed to keep the base of the tubes dry. In extreme cases, connections were coated with beeswax to insulate exposed surfaces and prevent current leakage.

Although stray light and humidity produce the same type of counting failures as insufficient light, adjustment of the photoelectric equipment must be carefully made, or, in remedying one condition, the other will be accentuated. The operation of the photoelectric relay depends upon the amount of current passing through the photoelectric tubes dropping below and rising above certain critical values. If the conditions under which the equipment is operating are such that the proper change of current does not occur when the light beams are blocked, the relay may fail to operate, giving no count, or it may not operate positively, causing the contacts to "chatter" and produce extra counts.

The solution of the problem required: (1) That the best possible adjustment of light beams be made; (2) that some provision be made to eliminate excess moisture and stray light where the machines were subjected to an exceptional amount of either; and (3) that the photoelectric equipment be adjusted to provide adequate factors of safety against failure by insufficient light and stray light or humidity.

It is apparent from the map in figure 7, which gives the locations of the 344 automatic traffic recorders now in operation, that this equipment is required to operate under wide ranges of temperature. Occasionally machines in the north are subjected to temperatures of 40° F. below zero, and in some desert regions to temperatures as high as 126° F. Like all machines required to operate under wide ranges of temperature certain precautions must be taken to insure satisfactory operation under extreme conditions. To prevent the stopping of the clock because of the congealing of oil in the rotor in cold weather, heaters are furnished and recommended for use whenever the machines are subjected to temperatures below freezing. However, cases have been found where machines have operated at 5° below zero without the aid of heaters. To accomplish this, the machines must be in perfect adjustment.

In one region it was found desirable and economical to use an insulated housing for the recorder. Experiments to determine the most economical combination of heaters to use under different temperature conditions showed that three heaters (one in each arm of the periscope, and one in the recorder box) would maintain a temperature difference of 42° F. between the recorder mechanism and the outside air, when the machine was enclosed in an insulated housing. This would be necessary only in subzero weather, and it is recommended that only the heaters in the periscope be used in moderately cold weather.

Extreme heat will also affect the operation of the recorder. The caesium on the cathode of the photoelectric tube will vaporize under temperatures in excess of 150° F., and sufficient current will flow through the tube, even when there is no light on it to prevent the operation of the photoelectric relay. Old tubes are affected in the same manner at temperatures less than 150° F. The recorder would be subject to such extreme heat only in exceptional cases. One State has reduced

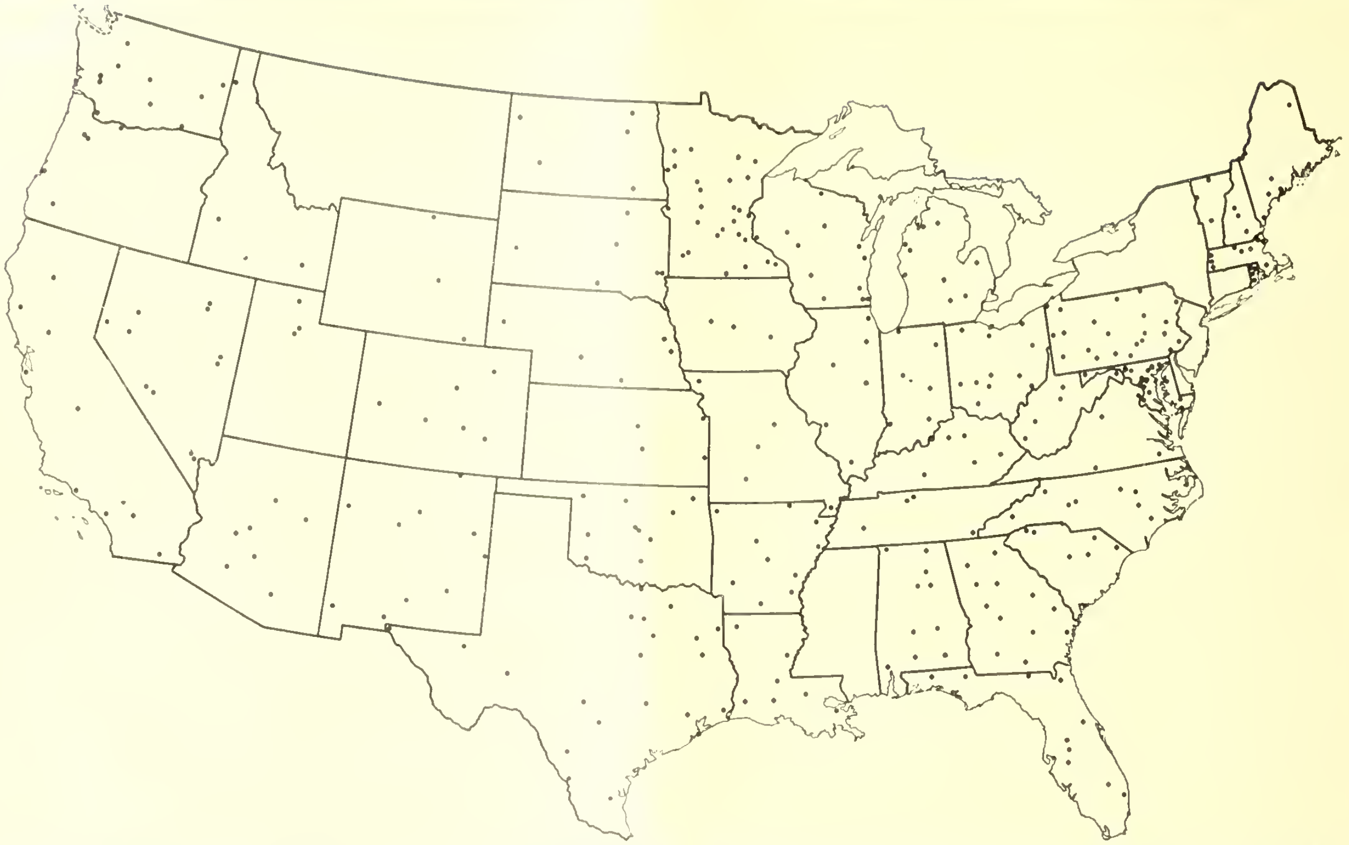


Figure 7.—Locations at Which Automatic Traffic Recorders Are Being Operated in Connection With the State-Wide Highway Planning Surveys.

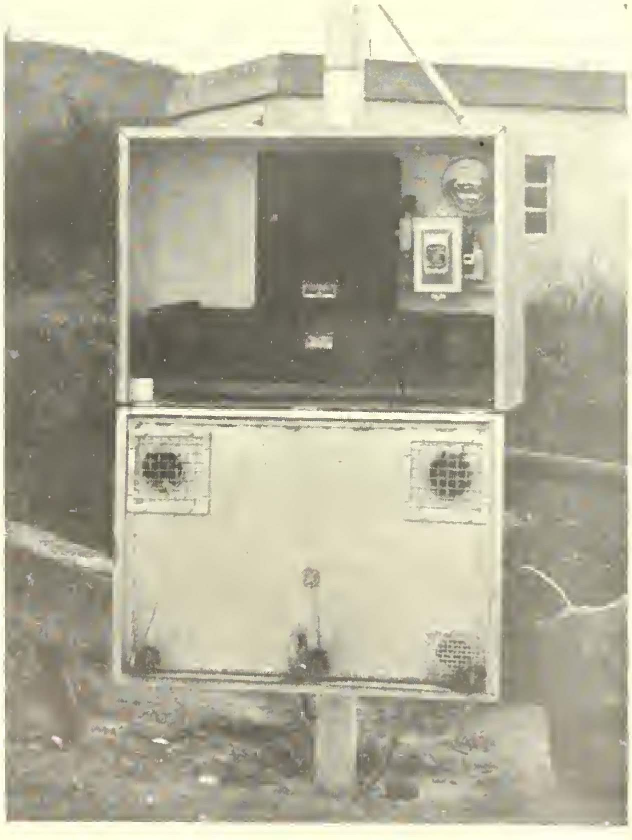


Figure 8.—Automatic Traffic Recorder Installed in an Aluminum Box for Protection Against Heat

the likelihood of difficulties from this source by enclosing the recorders in aluminum boxes. The box, as shown in figure 8, provides a dead air space which acts as insulation around the recorder, and the light-colored surface reflects rather than absorbs heat.

IMPROVED MAINTENANCE HAS INCREASED EFFICIENCY OF MACHINES

Heat also affects the infrared ray filters, causing them to fade because of the melting and running out of the gelatinous substance between the parallel glass plates. A new type of filter, consisting of a single glass plate, is now being used to replace the old filters when failures occur.

In one instance, a short time after a State had begun operating automatic recorders it was found that one of the machines was not recording traffic from late afternoon until morning. A check of the machine showed it to be in perfect working order, but when inquiry was made of people living in the vicinity, it developed that the recorder interfered with their radio reception and someone was blocking the light beams to prevent this disturbance. As soon as the State devised and installed a filter that eliminated radio interference, the recorder was no longer molested and produced satisfactory 24-hour records. Since that time provision has been made for elimination of radio interference, wherever necessary, at the time of installation.

In a few instances States installed automatic traffic recorders with the receiving units facing north to minimize the effect of stray light. It was found that during certain hours on bright days light-colored vehicles passed the machines without being counted. The phenomenon is accounted for by the fact that the light-colored vehicles will, during certain hours of the day, reflect enough light to replace the usual light from the light source of the recorder so that the machine fails to record the vehicle. However, the resulting errors were insignificant, because the time during which they occurred was limited and the number of light-colored vehicles on the highways was negligible. Any errors of this nature may be eliminated, however, by installing the receiving unit so that light cannot be reflected into the lens of the receiving unit by passing vehicles.

In the new models of automatic traffic recorders, all weaknesses in the early equipment have been considered and provisions are being made to eliminate them. In a counter recently developed the effects of both reflected and stray light are reduced by using an amplifying unit designed for use with "chopped" light. The chopped light is produced by placing the light source behind a shutter which is rotated at high speed. Reflected light will not replace the usual light supplied by the light source; and reflected light, therefore, cannot affect the operation of the counter.

The automatic traffic counters were probably the first photoelectric equipment ever installed to operate over a wide range of climatic conditions. To obtain satisfactory operation of the machines, it was necessary that each be adjusted for the particular range of conditions to which it was exposed. The making of these adjustments required a thorough knowledge of the operation of photoelectric equipment. Schools in a number of conveniently located cities were, therefore, conducted by the manufacturer who had the first contract to furnish machines. These schools were attended by representatives of all but two of the States in which machines were installed at that time. Performance improved so rapidly that the average operating efficiency of the machines throughout the country rose from approximately 66 percent to approximately 88 percent during the first month's operation after conducting the schools, and since that time the operating efficiency has been steadily increasing.

Improved methods of maintenance were developed at the schools with the result that good records have since been obtained at low cost. The method most generally used involves the appointment as supervisor of some person who has attended or has had special instruction similar to that given at the schools. The supervisor, who is usually assigned other duties in addition to those connected with the maintenance of this equipment, maintains a central headquarters and visits the machines only when notified by local representatives that the equipment is not operating properly. No other person is permitted to make repairs or replacements of any parts except burned-out bulbs, which may be replaced by certain local representatives of the State.

Although the supervisor is the only one permitted to make repairs, local representatives of the State, such as maintenance employees, State police, or other State employees who pass the recorder in regular line of duty, are instructed to observe the machines at such times to determine if they are functioning. If no click is heard when a vehicle passes, the main office is advised immediately and the supervisor is sent to make repairs. Arrangements are also made for some local employee to remove and forward the record tape to the central office once a week. This same employee is instructed in the method of making certain minor adjustments, and he also replaces the bulbs in the light source at periodic intervals to prevent failure caused by burned-out bulbs.

MACHINE COUNTS CHECK CLOSELY WITH MANUAL COUNTS

In maintaining the machines it should be remembered that while interruptions in operation which prevent the accumulation of records for considerable periods of time are serious, minor interruptions which cause the loss of only a few hours of record or even the occasional loss of a day or two of records will not seriously affect the value of the results obtained. In spite of short interruptions, enough information is usually available on which reliable estimates of the missing counts may be based so that the effect on the final results is negligible.

Some States that keep accurate records of the performance of their recorders have found that where machines are adequately maintained, the average operating efficiency of all machines in the State will exceed 95 percent. Individual machines frequently operate at 100 percent efficiency as long as 3 or 4 months at a time. Collisions, power and equipment failures, and other factors, any one of which will discontinue operation of the recorder, prevent an efficiency of 100 percent over an extended period of time.

There are two sources of error in results obtained by the photoelectric type of traffic recorder that should be mentioned. The simultaneous passage of two vehicles is recorded as one vehicle. Offsetting this, single vehicles with peculiar bodies or with large openings on the body, such as tractor-truck semitrailer combinations, are frequently recorded twice or more. These errors tend to compensate each other and check counts indicate that generally the machine count is within 1.5 percent of the actual traffic. Since the object in using automatic traffic recorders is to obtain records from which traffic trends can be determined, slight errors of this nature do not impair the value of the record.

It has been found that as a general rule automatic traffic counters record quantities in excess of actual figures. The accuracy of the automatic count also appears to vary with the volume of traffic, more accurate results being obtained on heavily traveled roads than on lightly traveled roads. The explanation for this is that the percentage of peculiarly shaped vehicles (farm machinery, etc.) is highest on lightly traveled roads, and also that more vehicles pass simultaneously in front of the recorder on heavily traveled roads, causing overcounts to be more nearly balanced by undercounts.

When properly adjusted, a machine may be relied upon to operate continuously with the same percentage of error, the amount of which is determined by the type of traffic peculiar to the location. In contrast, individuals are known to differ in their ability to count the number of vehicles passing a given point. These differences can be attributed to the fact that when an individual counts traffic for a considerable period of time, he cannot maintain the same degree of accuracy because of the monotony of the task.

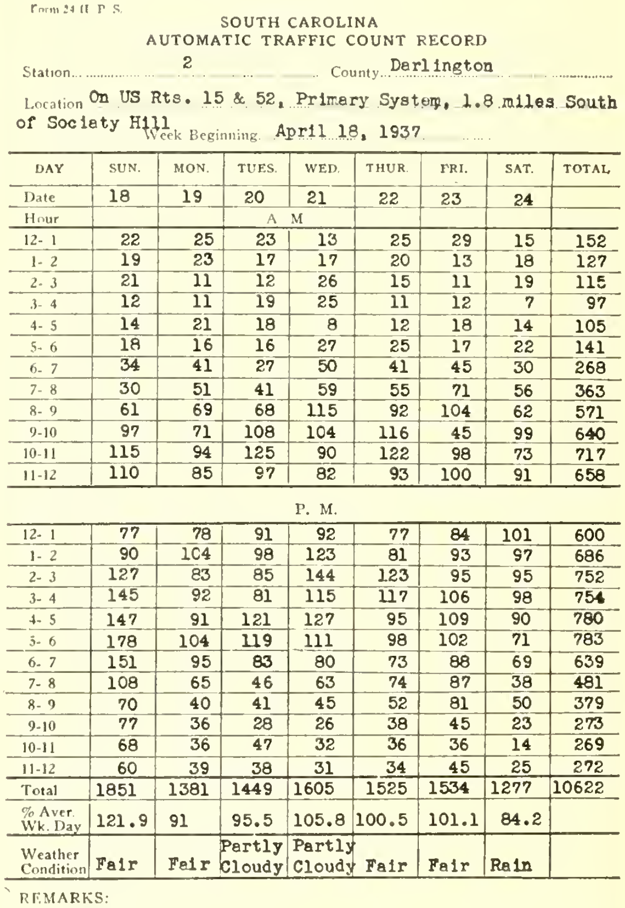
Table 1 shows the results of manual counts made to check a number of automatic recorders located at different points throughout the country.

TABLE 1.—*Comparison of automatic traffic recorder counts and manual counts of traffic made to check their accuracy*

Length of count	Count made— Manually	Count made— Mechanically	Difference
Hours	*Vehicles*	*Vehicles*	*Percent*
21	6,464	6,445	0.29
11	2,704	2,708	.15
13	2,001	1,998	.15
13	1,372	1,372	--
11	811	829	2.22
9	770	782	1.56
16	569	599	5.27
2	529	540	2.08
13	450	454	.89
9	371	386	4.04
3	344	342	.58
3	327	336	2.75
2	318	321	.94
4	224	224	--
2	219	221	.91
3	144	148	2.78

As a result of manual counts made in one State it was suggested that a special circuit be designed for use in automatic recorders located on roads carrying little traffic. This circuit would control the counting mechanism in such a manner that once both light beams were blocked and a count made, another count could not be registered until both light beams were again free. This would help to prevent overcounts on little-traveled roads. A special circuit for such use is already being developed. This circuit will make it possible to obtain more reliable counts on roads carrying a large number of horse-drawn vehicles.

In order to provide a convenient system for tabulating figures taken from machines, the record card shown in figure 9 was developed. On the back of the first weekly record card for each recorder a detailed sketch of its location is made. Under "Remarks" any unusual conditions that may affect traffic are entered. Data obtained directly from the tapes of properly operating recorders are entered in black ink. All estimated data for periods when the machine is known to be functioning improperly are entered in red ink. Thus, when an analysis of the records is made, the analyst has before him in a convenient form practically all of the information needed.



Form 24 U. P. S.

SOUTH CAROLINA
AUTOMATIC TRAFFIC COUNT RECORD

Station 2 County Darlington

Location On US Rts. 15 & 52, Primary System, 1.8 miles South of Society Hill

Week Beginning April 18, 1937

DAY	SUN.	MON.	TUES.	WED.	THUR.	FRI.	SAT.	TOTAL
Date	18	19	20	21	22	23	24	
Hour			A M					
12- 1	22	25	23	13	25	29	15	152
1- 2	19	23	17	17	20	13	18	127
2- 3	21	11	12	26	15	11	19	115
3- 4	12	11	19	25	11	12	7	97
4- 5	14	21	18	8	12	18	14	105
5- 6	18	16	16	27	25	17	22	141
6- 7	34	41	27	50	41	45	30	268
7- 8	30	51	41	59	55	71	56	363
8- 9	61	69	68	115	92	104	62	571
9-10	97	71	108	104	116	45	99	640
10-11	115	94	125	90	122	98	73	717
11-12	110	85	97	82	93	100	91	658

P. M.

12- 1	77	78	91	92	77	84	101	600
1- 2	90	104	98	123	81	93	97	686
2- 3	127	83	85	144	123	95	95	752
3- 4	145	92	81	115	117	106	98	754
4- 5	147	91	121	127	95	109	90	780
5- 6	178	104	119	111	98	102	71	783
6- 7	151	95	83	80	73	88	69	639
7- 8	108	65	46	63	74	87	38	481
8- 9	70	40	41	45	52	81	50	379
9-10	77	36	28	26	38	45	23	273
10-11	68	36	47	32	36	36	14	269
11-12	60	39	38	31	34	45	25	272
Total	1851	1381	1449	1605	1525	1534	1277	10622
% Aver. Wk. Day	121.9	91	95.5	105.8	100.5	101.1	84.2	
Weather Condition	Fair	Fair	Partly Cloudy	Partly Cloudy	Fair	Fair	Rain	

REMARKS:

FIGURE 9. SAMPLE WEEKLY RECORD CARD USED IN TABULATING DATA OBTAINED BY AUTOMATIC TRAFFIC RECORDERS.

COUNTERS PLACED AT CAREFULLY SELECTED LOCATIONS

The best method of estimating missing data for comparatively short periods will depend upon the particular circumstances. When the record for part of a day is lost, but the record for the rest of the day is adequate, an effort should be made to establish the relationship between the available data for that day and the data for the corresponding days in each of the two preceding and each of the two succeeding weeks. These relations can then be used to obtain a reasonably accurate estimate of the required data.

Ordinarily, the hourly distribution and volume of daily traffic from Monday through Friday is fairly uniform for a given road. Therefore, when data are missing for a short period on one of these days, a safe estimate can often be made by simply averaging the counts for the corresponding hours shown for the other 4 days of the same week.

To obtain the most valuable records by the use of automatic traffic recorders, it is essential that the recorders be properly located. In selecting locations the special uses to which the data are to be adapted must be considered. These uses generally are:

1. To aid in expanding sample counts that are obtained at other points on the highway system.

2. To provide a basis for analyzing various daily, weekly, seasonal, and other traffic distributions, or "patterns" in connection with determining the relative reliability of various methods of sampling.

3. To provide a basis for predicting future traffic.

In addition, there are a number of other uses which may be made of automatic traffic recorder data. Records obtained by these machines will: Provide a means of studying the relations between Sunday traffic volumes and weekday traffic volumes during the year; aid in studying the frequency distribution of daily traffic volumes; and assist in investigating traffic fluctuations produced by the opening of new roads or improvement of old roads. Specially equipped machines are now being used to study fluctuations within the hour.

To accomplish these purposes with a minimum number of machines, special care must be taken to locate the recorders so that each traffic pattern developed from a continuous record will be typical of a certain class of traffic. This suggests that the first step is to classify all roads according to the class of traffic carried; that is, tourist, farm-to-market, combination tourist and farm-to-market, metropolitan, etc. The next step is to pick typical locations for each classification at which it will be easiest to isolate the effect of the predominant class of traffic.

Probably the most important use of data recorded by automatic traffic recorders is to furnish a basis for establishing fundamental traffic trends and determining normal traffic patterns. These trends and patterns are of great importance in connection with predicting future traffic and in checking the dependability of various methods of sampling traffic.

Complete data for a considerable period of time are required to determine normal traffic patterns. A seasonal traffic pattern, for instance, is not merely a curve showing seasonal variations in traffic for a certain year, but it is a traffic pattern based upon counts made for several years. This pattern will indicate the usual seasonal variation together with the degree to which it may be expected to vary from year to year. Similarly, traffic patterns must be determined for days of the week, hours of the day, and in certain cases, intervals during each hour. A traffic pattern cannot be considered as fixed, as changes are constantly occurring.

Additional phases of this problem may be indicated by pointing out that in order to determine the normal seasonal traffic variation at a certain location, it is necessary to know the underlying growth tendency (secular trend) as indicated by records over a considerable number of years. Obviously the normal distribution of traffic by days of the week or hours of the day cannot be determined without, at the same time, determining seasonal variation. Once normal trends and patterns are established, it will be possible to isolate the influences of general and local economic conditions and often unusual factors influencing traffic patterns can be identified. Data collected by hours over a period of several years also serve as a basis for determining schedules for taking limited samples that will yield satisfactory results.

The greatest progress in ascertaining the influence of general and local conditions upon traffic can be made by keeping the machines at fixed locations for long periods of time. Therefore, automatic recorders should not be moved from one location to another unless the number of available recorders is so small that they can be used only to develop factors for the expansion of short counts. Expansion factors obtained in one year and applied to short counts obtained in another are of questionable reliability, especially where there is no reliable information regarding the secular trend of the traffic patterns on these roads.

COUNTER RECORDS USED IN EXPANDING MANUAL COUNT DATA

In connection with the problem of deriving factors to be used in expanding short manual counts obtained at many points, it is desirable to obtain answers to the following questions for each available set of data:

1. What is the relation between the number of vehicles passing during the period of the day when the manual counts are scheduled, and the number of vehicles passing during the entire 24-hour period?

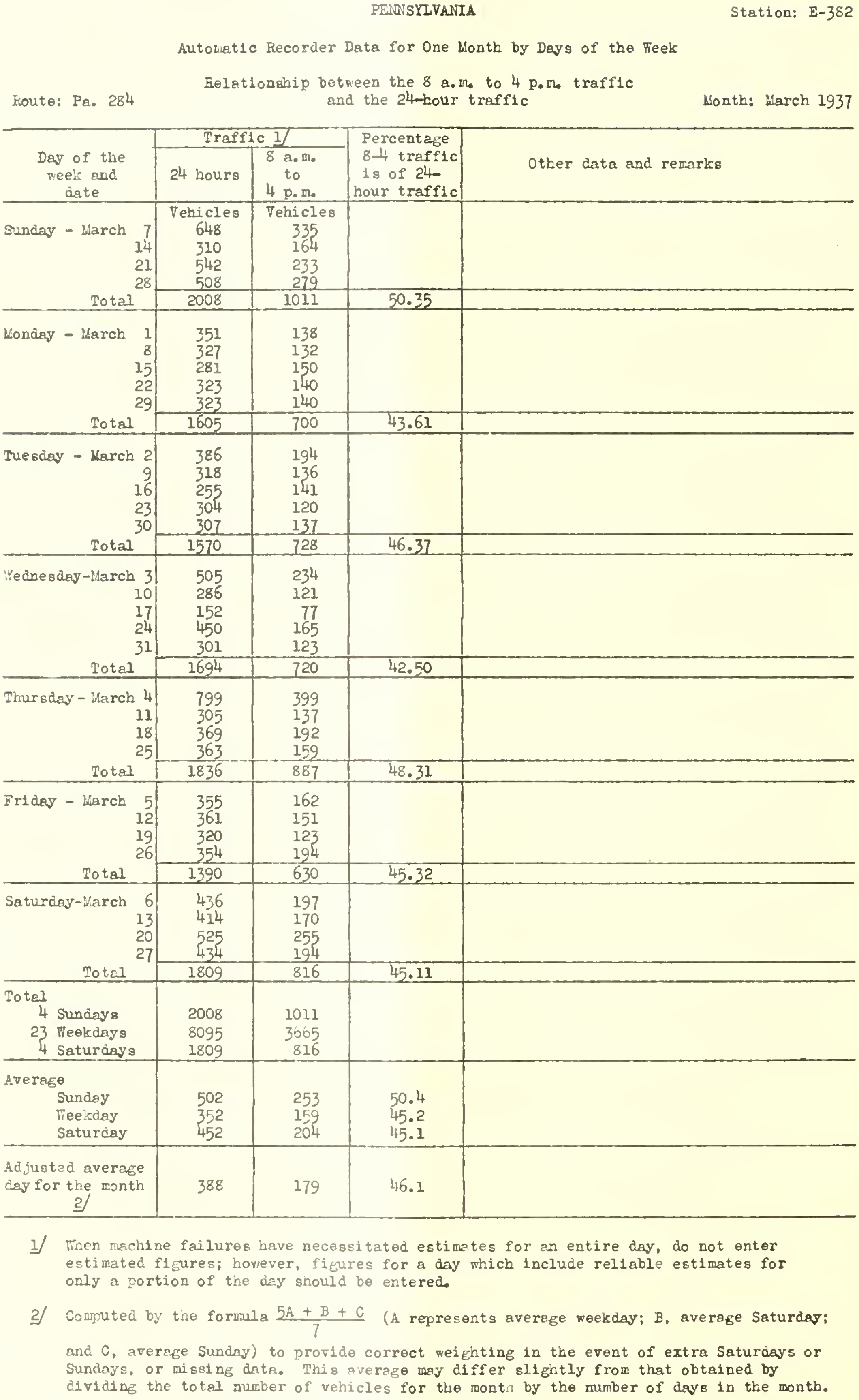
2. What is the relation between the average daily traffic from Monday to Friday and the average daily traffic for the whole week?

3. What are the relations between the average daily traffic for each month and the average daily traffic for the year?

Since these basic relations are desired for traffic passing each machine, it is convenient to arrange the data in forms similar to the partially filled forms shown in figures 10 and 11.

On different classes of roads, these relations are found to be quite different. By classifying the influences that are known to account in part for the variations in the relations, and by modifying the relations in accordance with the data obtained at manual control stations where a number of repeat counts are obtained at intervals throughout the year, a number of classified groups of expansion factors are obtained. From these groups of expansion factors, those applicable to a single route where short sample counts have been made can be selected and applied.

Arrangement of the data in the form shown in figures 10 and 11 also facilitates investigation of the relative reliability of various methods of sampling traffic. In studies of this type, it is assumed that counts have actually been made on the schedule being investigated, with the first count on the schedule beginning on the first day



PENNSYLVANIA

Station: E-382

Automatic Recorder Data for One Month by Days of the Week

Relationship between the 8 a.m. to 4 p.m. traffic and the 24-hour traffic

Route: Pa. 284

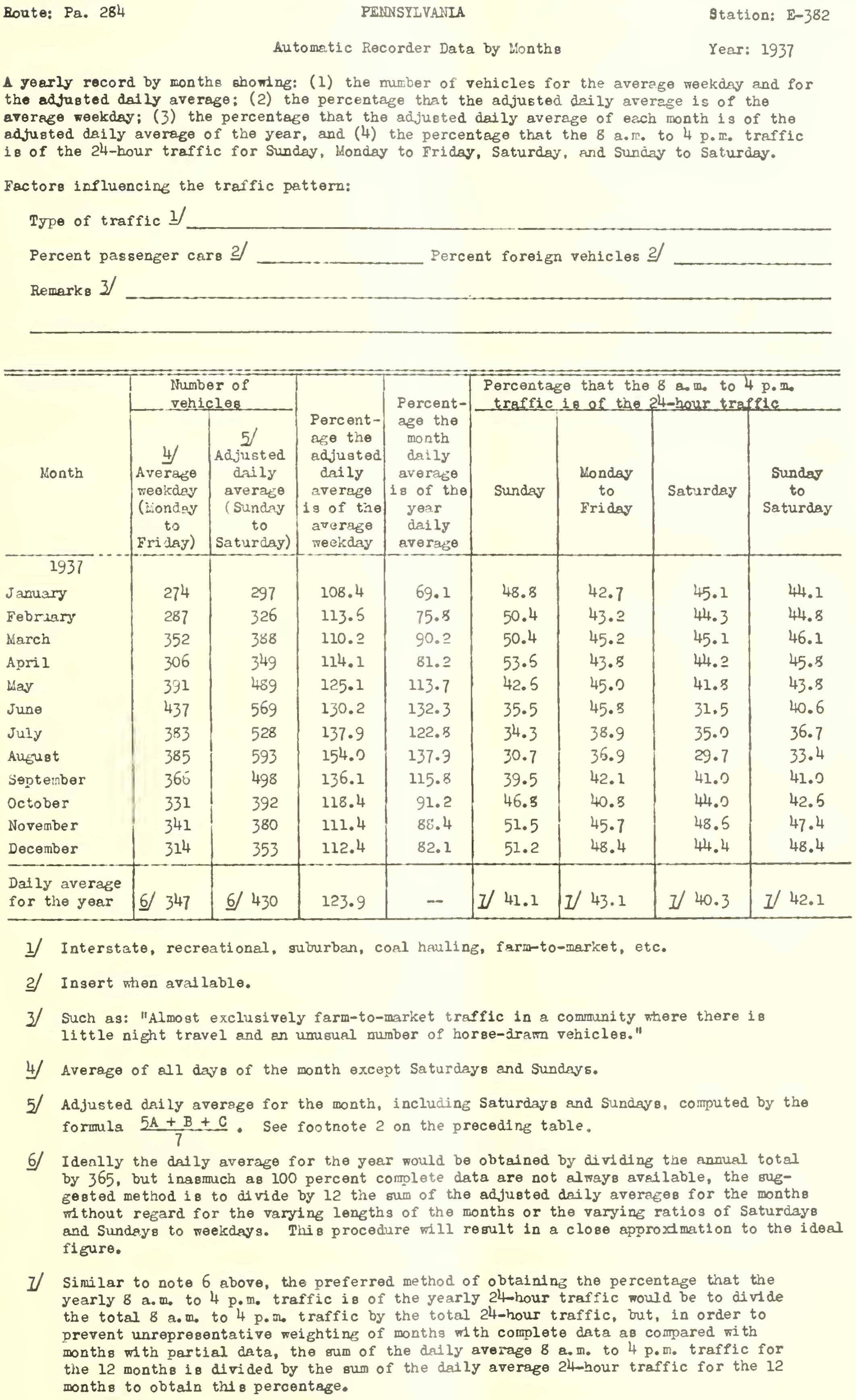
Month: March 1937

Day of the week and date	Traffic 1/		Percentage 8-4 traffic is of 24-hour traffic	Other data and remarks
	24 hours	8 a.m. to 4 p.m.		
	Vehicles	Vehicles		
Sunday - March 7	648	335		
14	310	164		
21	542	233		
28	508	279		
Total	2008	1011	50.35	
Monday - March 1	351	138		
8	327	132		
15	281	150		
22	323	140		
29	323	140		
Total	1605	700	43.61	
Tuesday - March 2	386	194		
9	318	136		
16	255	141		
23	304	120		
30	307	137		
Total	1570	728	46.37	
Wednesday-March 3	505	234		
10	286	121		
17	152	77		
24	450	165		
31	301	123		
Total	1694	720	42.50	
Thursday - March 4	799	399		
11	305	137		
18	369	192		
25	363	159		
Total	1836	887	48.31	
Friday - March 5	355	162		
12	361	151		
19	320	123		
26	354	194		
Total	1390	630	45.32	
Saturday-March 6	436	197		
13	414	170		
20	525	255		
27	434	194		
Total	1809	816	45.11	
Total				
4 Sundays	2008	1011		
23 Weekdays	8095	3665		
4 Saturdays	1809	816		
Average				
Sunday	502	253	50.4	
Weekday	352	159	45.2	
Saturday	452	204	45.1	
Adjusted average day for the month 2/	388	179	46.1	

1/ When machine failures have necessitated estimates for an entire day, do not enter estimated figures; however, figures for a day which include reliable estimates for only a portion of the day should be entered.

2/ Computed by the formula $\frac{5A + B + C}{7}$ (A represents average weekday; B, average Saturday; and C, average Sunday) to provide correct weighting in the event of extra Saturdays or Sundays, or missing data. This average may differ slightly from that obtained by dividing the total number of vehicles for the month by the number of days in the month.

FIGURE 10.—FORM USED IN SUMMARIZING EACH MONTH'S TRAFFIC DATA.



Route: Pa. 284 PENNSYLVANIA Station: E-382

Automatic Recorder Data by Months Year: 1937

A yearly record by months showing: (1) the number of vehicles for the average weekday and for the adjusted daily average; (2) the percentage that the adjusted daily average is of the average weekday; (3) the percentage that the adjusted daily average of each month is of the adjusted daily average of the year, and (4) the percentage that the 8 a.m. to 4 p.m. traffic is of the 24-hour traffic for Sunday, Monday to Friday, Saturday, and Sunday to Saturday.

Factors influencing the traffic pattern:

Type of traffic 1/ ____________________

Percent passenger cars 2/ __________ Percent foreign vehicles 2/ __________

Remarks 3/ ____________________

____

Month	Number of vehicles		Percentage the adjusted daily average is of the average weekday	Percentage the month daily average is of the year daily average	Percentage that the 8 a.m. to 4 p.m. traffic is of the 24-hour traffic			
	4/ Average weekday (Monday to Friday)	5/ Adjusted daily average (Sunday to Saturday)			Sunday	Monday to Friday	Saturday	Sunday to Saturday
1937								
January	274	297	108.4	69.1	48.8	42.7	45.1	44.1
February	287	326	113.6	75.8	50.4	43.2	44.3	44.8
March	352	388	110.2	90.2	50.4	45.2	45.1	46.1
April	306	349	114.1	81.2	53.6	43.8	44.2	45.8
May	391	489	125.1	113.7	42.6	45.0	41.8	43.8
June	437	569	130.2	132.3	35.5	45.8	31.5	40.6
July	383	528	137.9	122.8	34.3	38.9	35.0	36.7
August	385	593	154.0	137.9	30.7	36.9	29.7	33.4
September	366	498	136.1	115.8	39.5	42.1	41.0	41.0
October	331	392	118.4	91.2	46.8	40.8	44.0	42.6
November	341	380	111.4	88.4	51.5	45.7	48.6	47.4
December	314	353	112.4	82.1	51.2	48.4	44.4	48.4
Daily average for the year	6/ 347	6/ 430	123.9	--	7/ 41.1	7/ 43.1	7/ 40.3	7/ 42.1

1/ Interstate, recreational, suburban, coal hauling, farm-to-market, etc.

2/ Insert when available.

3/ Such as: "Almost exclusively farm-to-market traffic in a community where there is little night travel and an unusual number of horse-drawn vehicles."

4/ Average of all days of the month except Saturdays and Sundays.

5/ Adjusted daily average for the month, including Saturdays and Sundays, computed by the formula $\frac{5A + B + C}{7}$. See footnote 2 on the preceding table.

6/ Ideally the daily average for the year would be obtained by dividing the annual total by 365, but inasmuch as 100 percent complete data are not always available, the suggested method is to divide by 12 the sum of the adjusted daily averages for the months without regard for the varying lengths of the months or the varying ratios of Saturdays and Sundays to weekdays. This procedure will result in a close approximation to the ideal figure.

7/ Similar to note 6 above, the preferred method of obtaining the percentage that the yearly 8 a.m. to 4 p.m. traffic is of the yearly 24-hour traffic would be to divide the total 8 a.m. to 4 p.m. traffic by the total 24-hour traffic, but, in order to prevent unrepresentative weighting of months with complete data as compared with months with partial data, the sum of the daily average 8 a.m. to 4 p.m. traffic for the 12 months is divided by the sum of the daily average 24-hour traffic for the 12 months to obtain this percentage.

Figure 11.—Form Used in Summarizing Traffic Data by Months.

of the year. By advancing the date of initial operation successively to each day throughout a complete period, a group of samples can be obtained which, when expanded according to the best available methods, can be studied by various statistical methods to obtain some measure of the reliability of the sample. This same procedure can then be extended to other proposed sampling schedules. After the relative reliability of a number of schedules is known, future traffic surveys can be planned so as to obtain the desired information in the quickest and most economical manner.

Because the automatic traffic recorders have been in use but a short time, the records are as yet inadequate to enable determination of normal seasonal patterns, or the causes of deviations from the normal patterns. Such determinations will be necessary, however, in connection with the isolation of individual influencing factors. After the effects of each of a number of factors can be distinguished, it is hoped that roads can be scientifically classified into groups having like traffic characteristics. When this is done, many of the changes in the flow of traffic may first be identified, and later predicted.

Figures 12, 13, and 14 show typical traffic patterns developed from records obtained by automatic recorders.

COST OF OPERATING COUNTERS DISCUSSED

A recent survey disclosed the average cost of operating one automatic recorder at a rural location for a month to be as follows:

Overhead	$3. 69
Supervision	4. 04
Maintenance:	
Labor	6. 57
Subsistence	2. 48
Travel	8. 51
Power	4. 05
Supplies, etc	2. 20
Total	23. 81
Preparation of records [1]	7. 86
Grand total	39. 40

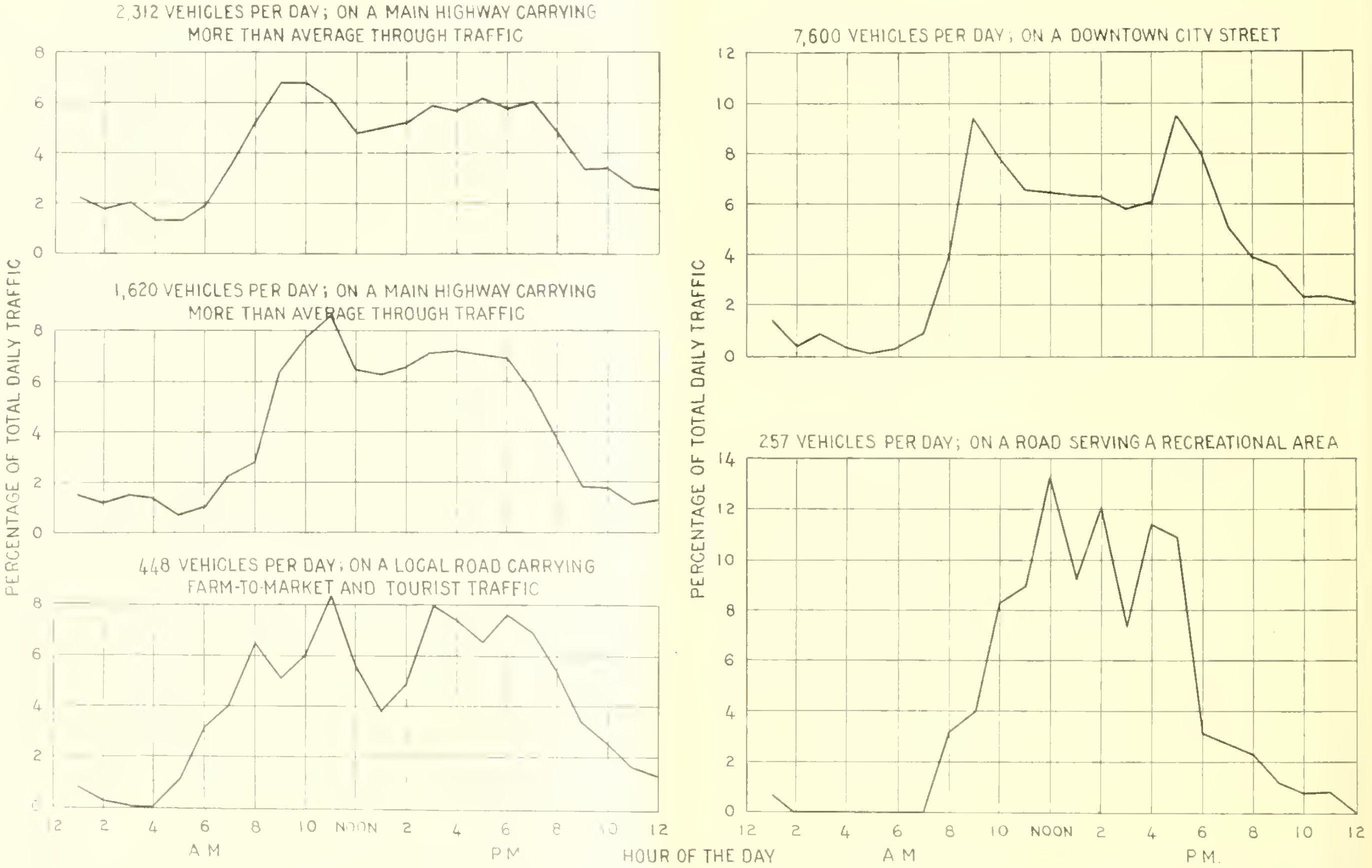


Figure 12.—Examples of Hourly Variations in Traffic Volume on Roads Carrying Different Classes of Traffic During 1937.

Costs of operating recorders at urban locations would probably be less because of reduced travel distance and lower power rates.

The average cost of installing one machine is $124.67. This cost varies widely, depending on the amount of travel required to reach the location.

The yearly cost of obtaining traffic counts by the automatic traffic recorders is small when compared

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[1] Preparation of records covers the cost of transferring figures from record tape to permanent card forms.

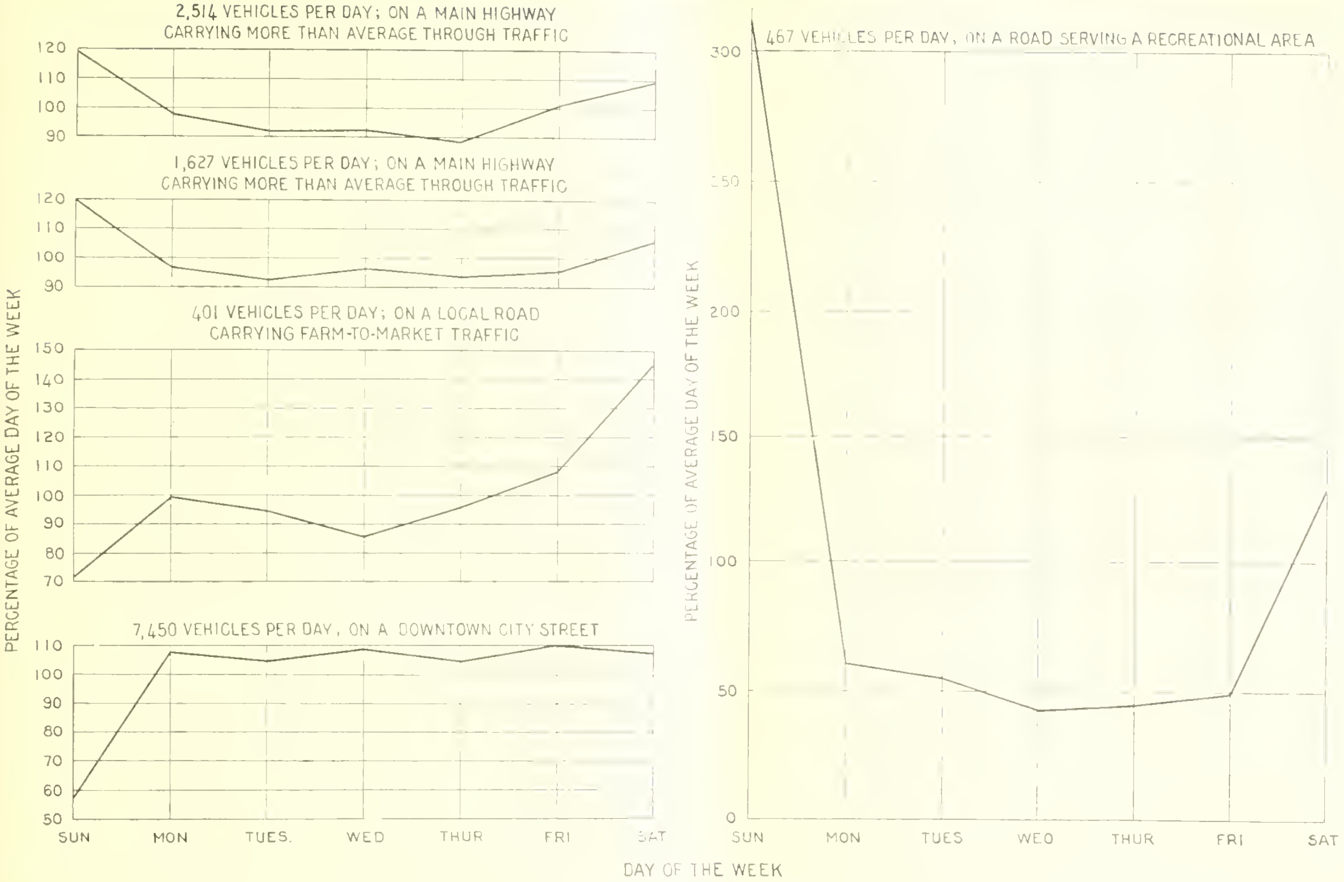

Figure 13.—Examples of Daily Variations in Traffic Volume on Roads Carrying Different Classes of Traffic During 1937.

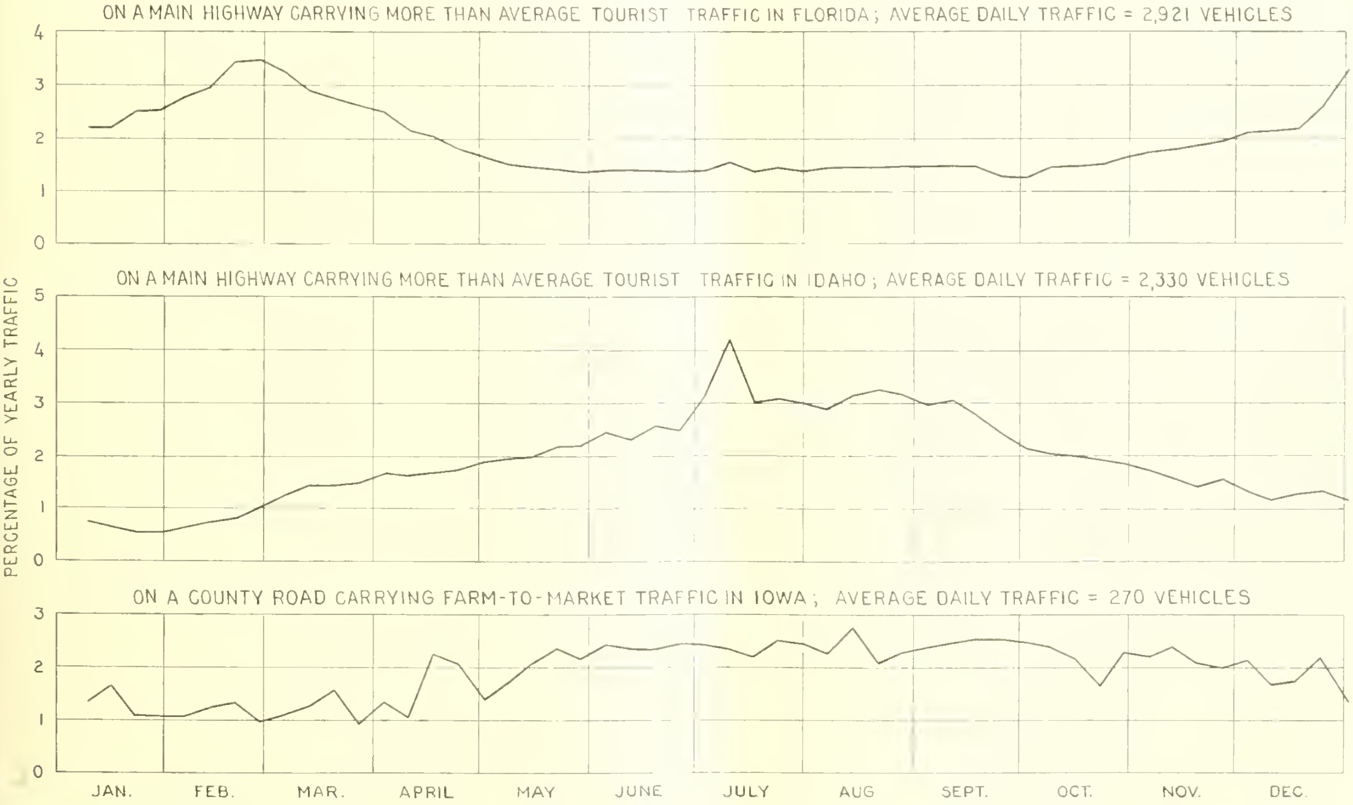

Figure 14.—Weekly Variations in Traffic Volume on Roads Carrying Different Classes of Traffic During 1937

with the cost of obtaining the same record manually. The following is an estimate of the annual cost of operating the average automatic traffic recorder:

Depreciation (based on assumed 5-year life and average first cost of $406.00)	$81.20
Installation (based on 5-year period and average installation cost of $124.67)	24.93
1 year's operation costs at $39.40 per month	472.80
Total	$578.93

The estimated yearly cost of obtaining the same record manually is between $6,022.50 and $8,212.50. This estimate is based on reliable figures, which show that the average cost of making an 8-hour count ranges between $5.50 and $7.50.

Before comparing the cost of counting traffic manually with the cost of automatic counting, it should be pointed out that a man stationed at an intersection can count traffic on all roads entering the intersection, whereas an automatic counter obtains a count for only one road. In addition, manual counts will give the composition and direction of traffic flow. However, as automatic traffic recorders are used to obtain continuous records, where the number of vehicles passing a point by time intervals is the information desired, and because only occasionally will more than one distinctive yet representative traffic pattern be obtainable at an intersection, the estimated cost of obtaining such records by the two methods may be compared. On the basis of these estimates, approximately 10 to 14 years' records could be obtained by the use of automatic traffic recorders for the cost of obtaining 1 year's manual record. As the number of automatic traffic recorders in operation increases and methods of inspection and collection of records are improved, the cost of operating each machine will probably decrease.

Considering the economy of the automatic traffic recorders, it might be asked why machines are not used exclusively. One reason is that in traffic surveys it is not enough to determine only the number of vehicles which use the highways. It is generally desired to classify the vehicles as passenger cars, trucks, tractor-trucks and semitrailers, and busses. Moreover, another classification distinguishes between those vehicles having license tags from States other than that in which they are observed, commonly called "foreign" vehicles, and those vehicles having license tags of the State in which they are observed, commonly called "local" vehicles.

Thus, a comprehensive traffic survey makes necessary numerous manual counts. However, it is believed that the most economical and desirable plan for conducting traffic surveys would be one in which most of the actual counting would be done by machines, and manual counting would be restricted to a minimum number of counts at a few control stations, all of which would be operated for the purpose of obtaining the necessary samples of classified traffic. Ratios between the various types of vehicles observed at the manual control stations could then be applied to all the vehicles counted by the machines.

ATTEMPTS BEING MADE TO DEVELOP PORTABLE COUNTERS

The automatic traffic recorder which has been described was designed for permanent installation to provide continuous records for long periods of time. The installation expense and electric power requirements of these machines make it impossible to extend their use to assist in accumulating the desired sample counts at thousands of points throughout a State, or in connection with a city traffic survey at hundreds of points within a city. Attempts to develop a satisfactory, low-cost, portable automatic recorder for this work have been in progress for a number of years. Efforts have been intensified since 1936 when many of the States began their planning surveys.

The most difficult problem encountered in the attempt to develop portable machines has been that of designing a suitable detecting unit that could operate economically as a self-contained unit. Efforts have been made to develop a suitable road switch, consisting of two pieces of spring steel, separated at intervals by spacers and encased in rubber. The road switch is placed on the road surface and is connected to a storage battery and counting unit. An electrical impulse actuates the counting unit each time the wheels of a passing vehicle press the spring steel strips together.

Only recently the Bureau has been experimenting with an air switch, which consists of a small rubber tube and diaphragm arranged to operate an electrical contact. The tube is placed on the road surface and, when compressed by the wheels of a vehicle, sufficient air is displaced to move the diaphragm. The movement of the diaphragm causes an electric contact to actuate a counting mechanism. This type of switch offers attractive possibilities.

Portable machines utilizing either the contact switch or the air switch may soon be available. However, experiments with such devices have been confined to paved roads. The problem of adapting them to unpaved roads still remains to be solved if these machines are to be of more than limited value in counting traffic. This consideration, of course, is unimportant in connection with city traffic surveys. The road switch detecting unit appears ideal for city traffic counts because of its ability to detect vehicles traveling in the same direction in different lanes, even when one vehicle lags only slightly behind the other. Also, whenever desired, road switches could be installed in a single traffic lane, or a group of machines could be installed, each of which would record traffic moving in a single lane.

A portable automatic recorder with a single-beam photoelectric detecting unit, operated by batteries, has recently been designed. It is expected that a model of this machine will soon be constructed and ready for testing. This machine offers particularly attractive possibilities for use on unpaved rural roads, and may be the means of effecting a considerable saving in the cost of future traffic surveys.

Numerous attempts have been made to develop detecting units of other types, including the use of hydraulic tubes on the road surface, sound equipment, and magnetic circuits, but thus far none of these attempts has proved successful.

The requirements of a satisfactory portable machine have been considered to be:

1. Continuous operation for 8 days without attention.
2. Reliable timing with error no greater than 5 minutes in 24 hours.
3. Efficient counting on both paved and unpaved roads.
4. Printing of the cumulative traffic total once each hour on the hour.
5. Installation to be accomplished by 1 man in about 15 minutes.

SUMMARY

Experience in the development and use of automatic traffic recorders indicates that they are a practical means of obtaining traffic records needed in planning future highway and street improvements. Present developments can be summarized as follows:

1. Three hundred and forty-four automatic traffic recorders located on all types of roads are now in use throughout the country, accurately counting and recording, hour by hour, all passing vehicles.
2. Records now being obtained from automatic traffic recorders are making available to traffic engineers, for the first time, complete traffic records for a large number of selected points which can be used in studying the fluctuations in traffic flow.
3. With a knowledge of the fluctuations in traffic flow, engineers can arrange future schedules for counting traffic to produce the most accurate results at minimum cost.
4. When records are obtained over a period of years, predictions of future traffic volumes and future traffic requirements can be made with much greater accuracy than is possible today.
5. A valuable basis is being provided for deriving and checking factors for expanding short counts being made in the current State-wide highway planning surveys.
6. Present developments in the field of portable automatic traffic recorders indicate that it is reasonable to expect that in the near future machines will be used extensively in making traffic counts.
7. Machines are well adapted to use in connection with city traffic surveys, and it is probable that they will be used extensively in cities in the future.

An Action Program to Advance Safety on the Highways

(*Continued from page 36*)

Government would be the establishment of an interdepartmental committee with representatives assigned from all of the Federal organizations which now have authority over, and responsibility for, elements of the safety problem under the Federal laws. In addition to this official organization there should be an advisory committee composed of representatives of the important national associations which are engaged in the traffic safety field in its broad aspects. This plan, together with the plan of organization of the State safety authority, would form a complete national structure for the formulation of an adequate nation-wide program and its effective operation.

8. *A national educational program.*—One of the most assuring steps that has been taken in the field of education is the action by the National Education Association and the American Association of School Administrators to prepare a yearbook on safety. This important undertaking will be supplemented by various bulletins and other useful material to be distributed to all the public schools of the country to serve as a background for sound safety instruction to school children. Too much emphasis cannot be placed upon the importance of this whole activity.

One of the most valuable contributions to a permanent safety program which will doubtless result from this organized study by the educators will be an evaluation and decision as to the supplementary safety activities which should be undertaken or continued on an even broader scale in connection with the public schools. One of the greatest hopes for improved safety conditions lies through the implanting of correct thinking and habits in those who are now passing through the public schools. Illustrative of these supplementary activities are the driver training and school patrol sponsored by the American Automobile Association. In addition, a widespread program of public education and of specialized training of traffic officers and engineers is under way by national organizations through funds provided by the Automotive Safety Foundation. This list of organizations is a notable one, and has been mentioned in connection with the formulation and recommendation of a safety program for each State.[2]

Every indication points to progress during the coming year in the safety movement far beyond that made in any previous year, but the concept of adequate governmental organization and comprehensive laws must be the objective continuously recognized as fundamental in the establishment of a permanent safety program.

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[2] These organizations include the following: American Automobile Association, American Legion, Automotive Safety Foundation, General Federation of Women's Clubs, Harvard Bureau for Street Traffic Research, Highway Education Board, International Association of Chiefs of Police, National Automobile Dealers Association, National Congress of Parents and Teachers, National Grange, National Safety Council, Northwestern University Traffic Safety Institute.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF APRIL 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$1,253,020	$616,510	65.3	$5,515,900	$2,757,095	264.9	$1,803,950	$901,220	20.1	$6,172,520
Arizona	2,386,118	1,682,950	109.3	1,364,414	923,937	74.8	772,235	420,301	37.4	1,879,053
Arkansas	2,956,064	2,917,364	180.0	1,206,419	1,194,729	79.9	75,203	74,606	4.6	4,345,009
California	6,776,131	3,711,531	150.1	8,403,393	4,421,759	147.1	4,140,304	2,216,853	106.8	3,316,338
Colorado	3,013,619	1,676,271	109.9	2,328,788	1,275,892	83.4	399,730	212,490	5.1	3,494,978
Connecticut	783,198	389,196	9.3	231,520	108,123	.7	1,160,550	570,541	11.5	1,729,010
Delaware	534,818	263,575	21.0	274,800	137,400	4.7	448,727	211,154	13.9	1,490,258
Florida	924,037	460,069	27.2	2,512,942	1,256,471	52.3	865,860	432,930	15.3	3,937,210
Georgia	2,120,355	1,018,985	123.4	6,756,057	3,378,024	318.7	1,667,760	833,880	81.0	6,557,782
Idaho	2,621,404	1,529,374	198.9	1,221,597	728,194	90.0	808,628	481,300	17.3	1,772,276
Illinois	10,675,567	5,201,039	309.8	9,039,189	4,442,023	179.2	3,618,550	1,809,200	77.0	4,680,102
Indiana	5,905,003	2,907,247	137.9	3,703,691	1,851,845	108.9	3,362,803	1,681,160	64.7	3,310,534
Iowa	6,937,712	3,192,716	226.0	5,571,442	2,529,856	165.0	1,800,583	727,242	54.8	2,844,929
Kansas	4,658,997	2,303,965	262.3	3,652,634	1,826,221	95.2	2,833,489	1,416,770	596.2	5,068,510
Kentucky	2,378,681	1,183,735	62.3	3,733,512	1,866,756	113.6	4,248,626	2,120,314	160.2	2,813,517
Louisiana	582,531	286,501	15.0	6,378,335	1,341,935	39.5	7,078,353	1,584,260	51.7	2,791,480
Maine	1,865,369	932,090	50.7	2,016,676	1,005,403	44.4	795,770	397,880	20.5	1,035,545
Maryland	930,330	465,145	13.4	1,814,594	905,534	28.9	460,360	230,180	.5	2,512,396
Massachusetts	4,417,322	2,208,660	20.3	1,976,395	988,197	4.1	619,286	309,498	4.6	3,307,942
Michigan	6,508,320	3,187,232	166.9	6,545,668	3,202,434	136.3	595,620	297,185	14.3	3,830,650
Minnesota	6,431,138	3,176,084	323.4	4,014,115	1,995,813	174.7	1,145,332	562,052	59.8	4,285,584
Mississippi	2,804,601	1,393,045	139.5	4,956,990	2,477,790	222.3	1,938,320	737,110	76.9	4,222,157
Missouri	8,527,179	4,154,997	450.7	5,149,144	2,527,800	154.5	3,801,530	1,597,774	131.0	4,343,668
Montana	3,740,954	2,093,009	277.7	2,276,138	1,279,948	106.1	139,957	78,723	10.8	4,601,919
Nebraska	2,606,553	1,292,467	271.1	5,044,399	2,509,383	449.9	4,301,948	1,520,660	246.7	3,400,408
Nevada	2,488,928	2,051,622	132.5	708,345	614,171	62.5	495,241	429,091	22.3	2,028,945
New Hampshire	360,810	177,963	6.5	937,460	465,122	16.0	469,863	202,010	8.8	1,201,201
New Jersey	2,055,959	935,415	20.3	2,762,676	1,380,133	19.9	129,000	64,500		2,769,460
New Mexico	4,300,658	2,640,385	290.9	1,567,329	1,055,938	73.6	552,340	336,750	77.1	1,918,786
New York	13,749,759	6,346,752	241.3	13,582,754	6,563,430	227.4	3,408,540	1,702,770	50.1	5,538,798
North Carolina	5,119,154	2,552,906	381.4	6,486,833	3,056,552	266.1	912,320	435,200	69.5	4,016,537
North Dakota	1,114,612	1,036,055	186.5	2,089,261	2,048,991	122.4	911,550	751,975	68.5	3,908,298
Ohio	4,515,291	2,177,861	58.8	8,836,712	4,387,910	99.0	1,640,600	820,260	13.8	9,235,680
Oklahoma	3,460,745	1,800,034	170.4	3,197,998	1,647,335	118.2	2,567,125	1,363,103	131.7	4,774,615
Oregon	3,938,162	2,308,406	121.9	1,834,370	1,103,345	95.2	750,347	429,580	21.7	2,569,108
Pennsylvania	12,903,979	6,407,944	176.3	7,572,135	3,768,776	121.8	1,948,209	973,067	35.9	7,023,406
Rhode Island	921,999	442,608	8.1	1,182,420	591,210	16.7	113,877	56,938	1.1	1,223,518
South Carolina	3,762,555	1,590,753	260.9	5,397,247	2,299,943	246.8	1,161,235	524,336	50.5	2,112,881
South Dakota	2,372,349	1,332,002	246.1	2,710,372	1,498,990	253.9	1,796,211	1,004,810	100.6	3,740,788
Tennessee	1,873,280	933,864	70.0	3,645,444	1,822,722	115.3	2,410,990	1,146,978	64.3	5,257,234
Texas	13,689,667	6,822,126	883.6	11,426,859	5,669,912	647.0	4,105,502	1,958,460	246.2	9,790,045
Utah	1,349,100	954,211	130.8	940,050	672,512	58.7	298,920	213,484	7.3	2,174,978
Vermont	1,052,720	505,707	29.6	1,679,730	739,095	43.1	132,807	61,184	.7	612,911
Virginia	3,312,069	1,646,536	154.2	5,454,263	2,686,455	153.1	1,243,376	597,712	43.9	2,441,324
Washington	2,309,729	1,201,310	75.6	3,962,820	2,081,530	50.3	1,192,618	622,913	33.0	1,544,781
West Virginia	1,476,181	741,784	42.1	1,575,497	1,045,843	46.7	768,617	447,392	21.2	2,816,062
Wisconsin	8,350,793	4,042,181	278.8	5,066,103	2,339,277	113.8	468,993	220,620	6.8	3,830,809
Wyoming	2,803,771	1,720,254	300.7	1,347,552	824,022	135.4	351,645	214,978	17.9	1,445,080
District of Columbia										
Hawaii	777,827	379,041	13.2	681,510	337,470	12.4	651,400	325,025	10.4	1,396,047
Puerto Rico				1,075,539	535,790	20.3	60,920	30,360	.2	668,225
TOTALS	190,399,118	98,991,477	8,001.9	191,410,031	96,169,036	6,274.7	77,425,720	36,358,779	2,986.2	171,783,292

U. S. GOVERNMENT PRINTING OFFICE: 1938

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF APRIL 30, 1938

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$4,151,115	$3,936,706	$3,887,955	136.9	$260,300	$260,300	7.8				$2,860
Arizona	2,569,841	3,163,876	2,531,293	193.7	38,548	38,548					
Arkansas	3,352,061	3,281,921	3,250,581	360.5	50,280	50,280		$49,746	$49,746	4.0	1,454
California	7,747,928	7,876,715	7,495,718	262.3	217,622	217,579	.3				34,632
Colorado	3,395,263	2,378,891	2,301,910	99.4	97,797	95,414	6.0				997,939
Connecticut	1,418,709	1,378,104	1,263,206	21.2	55,490	55,000	1.2	124,130	64,435	.2	36,067
Delaware	900,310	870,612	842,125	65.2	10,234	10,234	.2	26,712	17,000	4.4	30,951
Florida	2,597,144	2,606,631	2,526,144	99.1	38,957	38,957					32,043
Georgia	4,988,967	2,004,937	1,936,445	112.8	2,473,464	2,205,084	116.7	441,600	441,600	30.1	405,838
Idaho	2,222,747	2,274,551	2,167,058	185.9	33,341	33,341		19,098	19,098		3,250
Illinois	8,694,009	7,855,079	7,579,988	447.5	1,169,926	1,088,926	40.8				25,095
Indiana	4,941,255	5,284,060	4,810,764	238.0	49,000	49,000					81,491
Iowa	4,991,664	5,237,324	4,884,857	528.3	105,902	104,865	.3				1,942
Kansas	4,994,975	4,749,621	4,695,918	381.8	229,126	226,954	10.9	7,144	7,144	.1	64,959
Kentucky	3,726,271	3,625,014	3,413,535	355.0	274,897	274,897	3.5				37,839
Louisiana	2,890,429	2,930,067	2,588,240	165.6	239,168	200,933	1.5	97,087	97,070	10.4	4,186
Maine	1,676,799	1,628,405	1,605,203	74.7	62,149	62,149	1.7				9,447
Maryland	1,750,738	773,435	766,648	27.2	547,536	547,536	11.2	235,866	218,812	4.8	217,742
Massachusetts	3,262,885	2,222,219	2,221,591	18.2	1,939,136	980,003	1.0				61,291
Michigan	6,301,414	6,663,691	5,945,844	288.6	284,921	284,921	3.3	223,938	63,746	.3	6,903
Minnesota	5,277,145	6,417,998	5,175,372	901.8	84,950	79,000					22,773
Mississippi	3,457,552	3,233,802	3,228,502	226.5	169,450	168,410	9.7				60,641
Missouri	6,012,652	5,290,265	5,208,397	776.9	785,410	730,708	.6	34,390	32,294		41,253
Montana	3,676,416	3,597,114	3,564,349	200.7	95,385	95,385	.1	8,462	8,462		8,220
Nebraska	3,870,739	3,433,440	3,321,591	362.3	446,353	446,353	8.3	70,270	70,270	1.8	32,525
Nevada	2,243,074	2,321,908	2,204,928	110.1	84,970	38,146	1.7				
New Hampshire	945,225	880,906	843,253	37.7	121,092	101,301	5.7				671
New Jersey	3,129,805	1,611,294	1,539,963	31.1	1,559,926	1,559,926	4.4				29,916
New Mexico	2,871,397	2,812,672	2,806,612	213.7	43,071	43,071		14,681	12,196		9,518
New York	11,046,377	10,777,954	10,272,533	170.5	43,700	43,700	.2	231,783	231,783	1.2	498,360
North Carolina	4,720,173	4,609,836	4,539,338	284.6	120,761	120,761	5.4	17,090	9,816		50,259
North Dakota	2,867,245	2,481,905	2,433,968	377.9	367,530	367,529	33.6	30,870	30,870	5.1	34,878
Ohio	7,670,815	6,967,085	6,851,506	294.9	703,332	697,332	3.7				121,978
Oklahoma	4,580,670	4,553,518	4,309,166	400.8	239,470	239,470	7.6	17,000	17,000	1.6	15,034
Oregon	3,038,642	3,189,864	2,969,387	164.4	45,580	45,580		11,846	11,846	1.0	11,829
Pennsylvania	9,347,797	8,024,397	7,438,451	262.7	1,999,961	1,743,728	20.0	19,481	19,481	.2	146,137
Rhode Island	989,208	1,113,140	989,208	18.8							
South Carolina	2,702,012	2,381,649	2,148,589	226.2	551,692	498,752	23.4	9,664	9,664	1.5	45,007
South Dakota	2,976,454	2,663,066	2,657,877	482.9	288,937	288,937	22.0	11,800	11,800		17,840
Tennessee	4,192,460	3,535,127	3,471,384	135.3	713,268	713,268	18.0				7,808
Texas	11,989,350	12,730,037	11,604,156	1,108.6	377,632	339,431	9.1				45,763
Utah	2,067,154	2,164,325	1,923,078	205.9	112,055	112,055	.1				32,020
Vermont	924,306	1,060,594	902,441	23.2	13,865	13,865					8,000
Virginia	3,652,667	3,484,973	3,285,362	938.6	287,731	284,434	14.7				82,871
Washington	3,026,161	3,364,831	2,940,420	164.2	65,824	65,824	.7				19,916
West Virginia	2,231,412	2,109,240	1,963,718	86.4	275,023	267,639	14.1				55
Wisconsin	4,823,884	5,217,570	4,734,584	343.4	91,394	89,300	.3				
Wyoming	2,219,155	2,173,387	2,165,009	152.4	33,287	33,287					20,859
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	955,853	871,389	17.4				62,530	54,644	.6	
TOTALS	195,000,000	184,849,609	174,029,050	12,790.6	17,899,443	16,052,113	409.8	1,765,188	1,498,777	67.3	3,420,060

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF APRIL 30, 1938

STATE	APPORTIONMENT	COMPLETED					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			Estimated Total Cost	Works Program Funds	NUMBER			
				Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,812,416	$ 3,801,471	49	1	12	$ 285,519	$ 219,219	4								$ 13,927
Arizona	1,256,099	1,300,469	1,216,144	15		5	18,841	18,841									21,114
Arkansas	3,574,060	3,296,774	3,288,785	56	6	31	236,101	235,211	2		2	$ 41,806	$ 41,806	1			8,258
California	7,486,362	7,345,501	7,186,523	47	8		112,295	112,295				10,000	10,000	5			177,544
Colorado	2,631,567	2,319,850	2,249,587	27	3	21	355,944	355,944	3								26,036
Connecticut	1,712,684	479,557	479,463	3	1	1	1,157,071	1,135,710	7								97,511
Delaware	418,239	130,000	128,144	1			279,052	279,052	2								11,043
Florida	2,827,883	2,475,666	2,457,348	31	5	6	57,888	57,888			7	178,690	178,690	1		63	133,956
Georgia	4,895,949	832,305	823,283	18	7	19	1,561,189	1,561,189	24	8		942,260	942,260	12	2	38	1,569,217
Idaho	1,674,479	1,393,286	1,364,907	20	3	12	252,696	252,696	2		6	4,261	4,261			3	52,615
Illinois	10,307,184	8,786,475	8,653,162	65	8		1,601,134	1,594,730	11			47,000	18,000	2			41,292
Indiana	5,111,096	4,774,115	4,632,083	42	15		450,720	450,720			162						28,293
Iowa	5,600,679	4,566,124	4,462,921	98	9	8	1,092,875	1,089,000	8								48,758
Kansas	5,246,258	4,403,154	3,977,319	55		5	1,197,631	1,178,217	3	1	2	112,152	19,148	2		1	71,574
Kentucky	3,672,387	1,357,377	1,336,725	19	5		1,984,219	1,694,487	5	6		612,579	612,579	3	1		28,596
Louisiana	3,213,467	1,881,670	1,872,707	19	2		667,588	667,559	7			644,918	595,438	4	1		77,763
Maine	1,426,861	1,268,330	1,261,145	19	1	4	160,788	142,540	1	1							23,176
Maryland	2,061,751	701,588	696,981	5	4	18	918,674	868,674	5	1	11	320,163	320,163	2	1	1	175,933
Massachusetts	4,210,833	2,993,173	2,990,739	22	4		865,680	865,680	4			249,991	249,991	1			104,423
Michigan	6,765,197	6,864,894	6,560,765	44	8		101,477	101,477				120,000	102,955	1	1		
Minnesota	5,395,441	4,815,817	4,769,278	83	13	50	590,070	584,020	3								42,143
Mississippi	3,241,475	2,570,838	2,568,720	53	6	16	385,300	385,300	6	1	2	119,100	119,100	2		5	168,355
Missouri	6,142,153	4,734,413	4,565,049	42	1		1,566,960	1,566,960	7		1	1,650	1,650			1	8,494
Montana	2,722,327	2,536,719	2,506,972	37	7		245,576	185,931	1	1		9,684	9,684		1		19,741
Nebraska	3,556,441	2,886,587	2,836,606	76	3	24	468,259	468,259	5			194,329	194,329	3			57,248
Nevada	887,260	877,374	864,025	8	3		13,308	13,308				4,257	4,257			5	5,670
New Hampshire	822,484	791,275	791,208	9	4	6	53,297	29,170	1			2,106	2,106			1	
New Jersey	3,983,826	3,095,189	3,087,143	20	6	1	812,334	812,334	4	3		44,730	44,730		1		39,619
New Mexico	1,725,286	1,700,885	1,693,233	19	1		25,879	25,879				3,129	3,129			1	3,045
New York	13,577,189	11,676,570	11,347,946	35	47		1,939,150	1,939,000	10	3		94,000	94,000				196,243
North Carolina	4,823,958	3,507,998	3,488,045	50	18		1,081,059	1,081,059	12		115	118,900	118,900	2			135,954
North Dakota	3,207,473	2,748,506	2,738,428	55	4		389,012	389,012		1							80,033
Ohio	8,439,897	2,520,630	2,399,382	19	7	13	6,249,227	5,766,081	40	1		20,000	20,000			1	254,435
Oklahoma	5,004,711	3,930,018	3,910,607	60	9	40	1,141,944	1,008,844	5	3		1,350	1,350			1	83,910
Oregon	2,334,204	2,294,365	2,240,272	16	6	2	74,215	74,215	1								19,717
Pennsylvania	11,483,613	9,075,239	8,362,371	71	18	9	2,850,615	2,683,567	15	5		150,473	150,473	5			287,202
Rhode Island	699,691	701,817	692,910	4	3												6,781
South Carolina	3,059,956	2,142,194	2,112,048	40	12	38	496,874	496,874	5	4	6	134,706	134,706	1	1	4	316,328
South Dakota	3,249,086	2,818,322	2,816,055	60	5	53	175,186	175,186	6			269,350	210,392	17			47,452
Tennessee	3,903,979	2,623,353	2,592,497	40	3	39	1,032,200	1,032,200	6	1	1	95,370	95,370	1			183,912
Texas	10,855,982	9,967,037	9,952,950	127	14	121	478,460	478,459	1	1	13	324,540	324,540	1		15	100,032
Utah	1,230,763	1,203,377	1,190,847	17	1	1	18,461	18,461									21,455
Vermont	729,857	751,992	705,786	10	7	20	10,900	10,900									13,171
Virginia	3,774,287	3,074,819	2,928,040	47	19	23	783,978	783,978	6	1		4,506	4,506			1	57,763
Washington	3,095,041	2,973,671	2,937,098	22	12	11	91,167	89,782	1								68,161
West Virginia	2,677,937	1,559,891	1,557,923	14	3	6	1,037,711	1,037,211	12			78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,471,868	4,437,324	37	6	10	402,360	402,157		1	11	97,748	97,748	1		13	85,454
Wyoming	1,360,841	1,214,014	1,203,724	13			111,212	111,212	1								45,905
District of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			179,710	169,698	2								
TOTALS	196,000,000	154,950,172	151,431,498	1745	328	625	36,061,806	34,700,186	238	43	339	5,052,603	4,805,116	69	9	155	5,063,200

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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 4 JUNE 1938

A SECTION OF ROAD IN MERIWETHER LEWIS NATIONAL PARK

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PUBLIC ROADS ►►► *A Journal of Highway Research*

Issued by the
UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
D. M. BEACH, *Editor*

Volume 19, No. 4
June 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

REPORTS SUBMITTED TO INTERNATIONAL ROAD CONGRESS BY REPORTERS FOR THE UNITED STATES

THE EIGHTH CONGRESS of the Permanent International Association of Road Congresses convened at The Hague, Netherlands, on Monday, June 20, 1938 and closed on Saturday, June 25, 1938. Papers written by reporters for the United States dealt with the following subjects in the agenda of the Congress

FIRST SECTION (CONSTRUCTION AND MAINTENANCE)

First topic:
  Progress accomplished, since the Congress at Munich, in the use of—
    a. Cement on road surfaces.
    b. Brick surfaces.
Second topic:
  Progress accomplished, since the Congress at Munich, in the preparation and use on roads of—
    a. Tar.
    b. Asphalt.
    c. Emulsions.

SECOND SECTION (TRAFFIC, IMPROVEMENT, AND ADMINISTRATION)

Third topic:
  Accidents on the road.
    a. Bases of statistics and their unification.
    b. Determination of causes of accidents, and means for diminishing them.
Fourth topic:
  Segregation, on the route, of the different means of transportation: Roads (one and two lanes), bicycle tracks, footpaths, parking spaces and exit roads, crossings and crossroads.
    a. Study of the circumstances under which these should be recommended or advised against.
    b. Application to automobile routes.

FIRST AND SECOND SECTIONS (COMBINED)

Fifth topic:
  Study and standard of quality of a road surface from the viewpoint of—
    a. Its slipperiness or ruggedness, and its resistance to skidding.
    b. Its property of reflecting or absorbing light (under artificial lighting).
Sixth topic:
  Study of road subsoils:
    a. Determination of properties of subsoils, testing methods, equipment.
    b. Influence of these properties on road construction (base and surface) and on road maintenance.

Reports submitted to the Congress by reporters for the United States will be published in PUBLIC ROADS. Papers dealing with the first and second topics are presented in this issue. The July issue will contain papers dealing with the third and fourth topics, and the August issue will contain papers dealing with the fifth and sixth topics.

PROGRESS IN THE USE OF CEMENT IN ROAD SURFACES

Reported by V. L. GLOVER, Engineer of Materials, Division of Highways, Illinois Department of Public Works and Buildings; Principal Reporter

I. CONCRETE PAVEMENT

Introduction.—The advancement in concrete pavement technique, as a rule, is gradual. New ideas, whether they pertain to design of pavements, materials, construction methods, or equipment, frequently must pass through an extended period of development before they become generally accepted. In the following are included only those which appear promising or have gained general acceptance during the last few years.

Subgrade.—More attention than ever before is being given to the selection and compaction of soils to prevent subsidence of fill material under pavement slabs. In some instances, unsatisfactory subgrade materials are removed and replaced with satisfactory materials. Compaction at the optimum moisture content of the soil by the "Proctor Method" [1] is rapidly gaining favor. The sheepsfoot roller is rapidly becoming the accepted type of mechanical equipment for use in compacting embankments.

Where subgrade soils change volume appreciably from absorbed moisture, it has been found that detrimental distortion of the pavement can be prevented by controlling the moisture content and density of the soil ahead of the paving. The moisture content and density desired are approximately equal to the optimum moisture content and maximum density as obtained by the Proctor Method and may be determined by swell tests on soil pats for any soil on a specified project.

Mechanical equipment for cutting and shaping the subgrade has been improved and is coming into increasing use. Machines have been developed which can cut to a depth of from 3 to 5 inches and move the excavated material by belt conveyors to the shoulders. In some instances the conveyors are designed to permit the loading of excavated material directly into trucks.

Side forms.—Because of the increased weight of finishing and subgrade equipment operating on the side forms, there has been a decided tendency toward the use of heavier forms. The prevailing practice requires a minimum base width of 8 inches for forms which are 8 inches or more in height, and there is a tendency to specify a base width at least equal to the height of the form. Power-driven machines known as form graders, designed to cut a true subgrade on which to set the forms and so give them better and more uniform support, are in common use, and mechanical

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[1] Engineering News-Record, August 31 and September 7, 21, and 28, 1933.

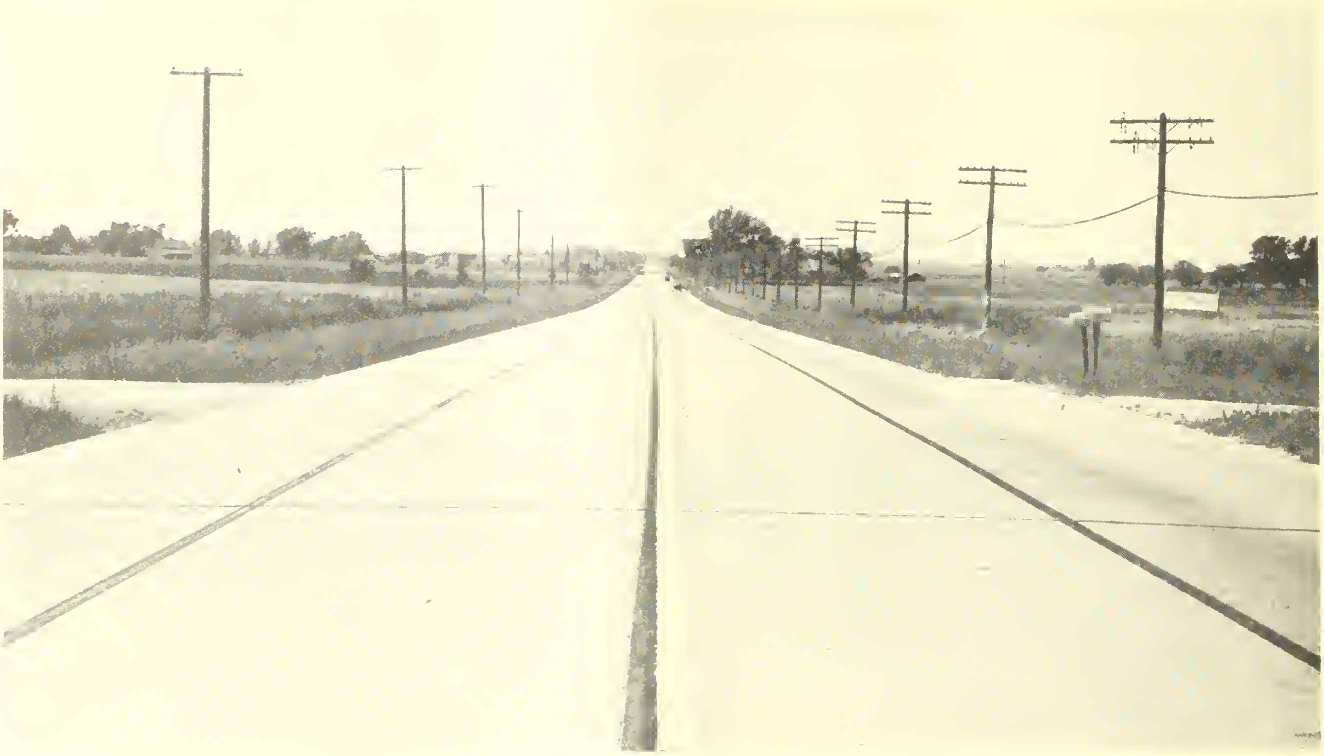

Figure 1.—Typical 4-Lane Concrete Pavement.

equipment for tamping soil under the forms has been successfully employed.

Pavement cross section.—The thickened-edge pavement design is generally used. The increase in thickness of the edges is usually accomplished in a uniform slope from the center thickness to the edge thickness in the outer 2 to 4 feet of the cross section. Occasionally the edges are of a uniform thickness over the outer 1 or 2 feet of the cross section and the edge thickness is reduced in a uniform slope to meet the center thickness at from 3 to 4 feet from the edges.

There is a marked tendency toward the selection of a pavement design for any given project on the basis of traffic requirements, taking into consideration the volume and weight of traffic loads and the condition of the subgrade. Where, in the past, only one standard pavement design was used, regardless of frequency and weight of wheel loads, two or more standard designs are often employed to suit the different traffic and subgrade conditions encountered. For normal heavily traveled rural highways common practice supplies a section 9 inches thick at the edges and 6 inches thick at the center, for heavy-duty pavements 10 inches at the edges and 7 inches at the center. Where the subgrade has high supporting value and traffic is fairly heavy 8 inches at the edges and 5½ inches at the center may be used, but in no case are sections less than 7 inches at the edges and 5 inches at the center permitted.

Of the methods available for computing stresses to determine the required cross section for given traffic loads and subgrade conditions, the mathematical analysis developed by Dr. H. M. Westergaard, Dean of the Graduate School of Engineering, Harvard University,[2] is undoubtedly the most adaptable to the variable conditions encountered. With proper selections of constants, and taking into account that wheel loads are dynamic forces, stresses may be reliably calculated by this method, which, however, like others available, does not apply directly to the calculation of stresses in thickened-edge designs.

By comparing section moduli of the most heavily stressed section of the pavement corners of thickened-edge designs with those of the corners of pavements of uniform thickness, an approximately "equivalent uniform thickness" may be determined for corners of any thickened-edge design, and Dr. Westergaard's method may then be applied. Similar procedure cannot be followed in the case of an unbroken edge of a thickened-edge pavement. Dr. Westergaard's method applies directly to the design of the interior thickness. In pavements where adjacent corners are mutually supported by dowels or other devices, and where increased edge thickness is obtained in the outer 2 feet, a fairly balanced design is obtained, as far as load stresses are concerned, when the center thickness is about two-thirds of the edge thickness.

An empirical method of determining the "equivalent uniform thickness" of corners of thickened-edge designs was developed by Frank T. Sheets, Consulting Engineer, Portland Cement Association,[3] who also suggests empirical formulas for design.

A pavement design balanced as to load stresses alone is strictly sufficient only in case of very short slabs. Recent investigation [4] indicates that the value of the thickened edge from the standpoint of load stresses may be entirely nullified in long slabs by increased warping stresses, and that warping stresses become unimportant only in a very short slab length, possibly as short as 10 feet. Whether these facts will affect materially the trend in pavement design remains to be seen.

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[2] Public Roads, Vol. 7, No. 2, April 1926; and Vol. 14, No. 10, December 1933.

[3] Concrete Road Design, Simplified and Correlated with Traffic (3d Edition), published by the Portland Cement Association, Chicago, Ill.

[4] Public Roads, Vol. 16, No. 10, December 1935.

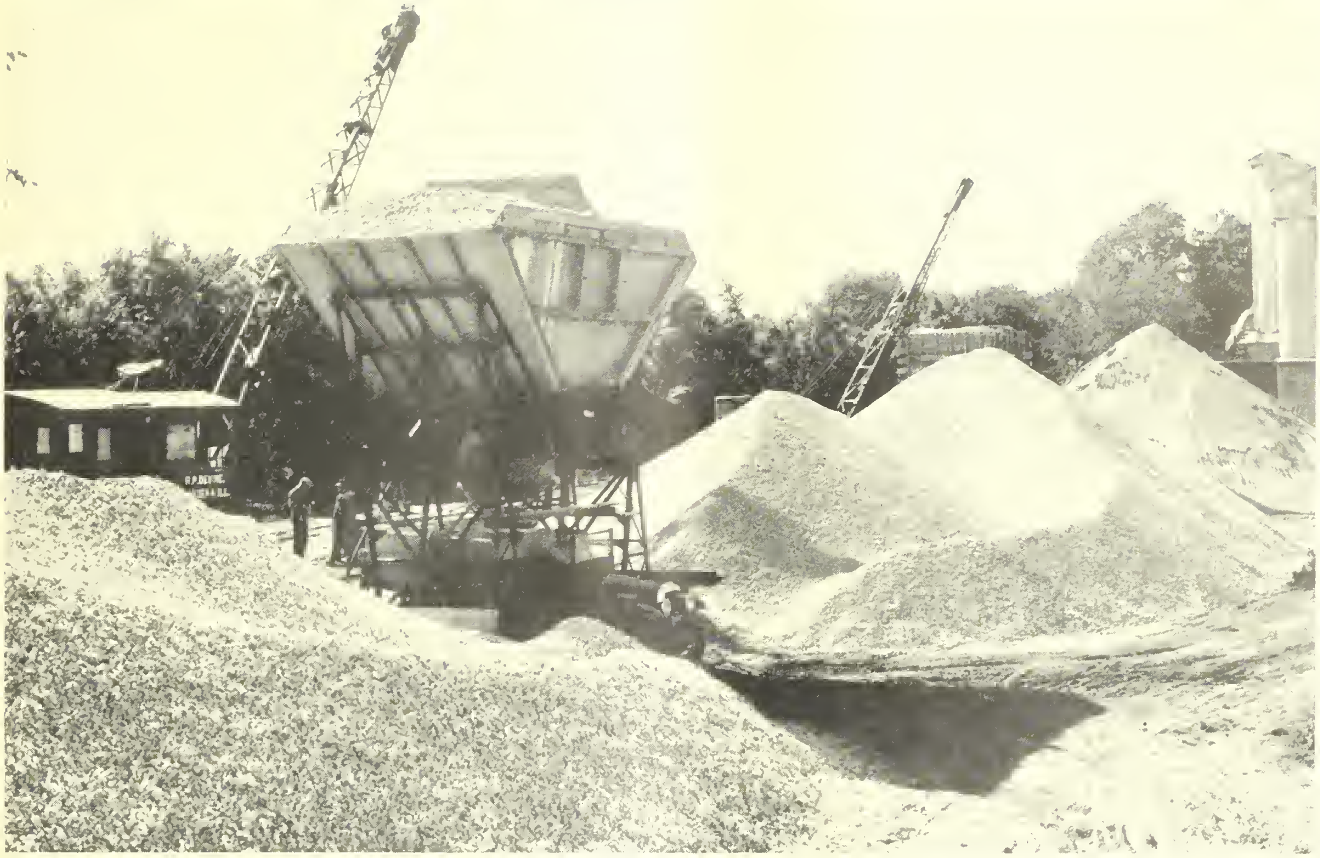

FIGURE 2.—PLANT FOR PROPORTIONING SEPARATED SIZES OF COARSE AGGREGATE.

The width of traffic lanes is being increased from 9 feet to as much as 12 feet because of the increase in speed of motor vehicles and the increase in width of the larger units. This additional width of lane may be expected to increase the capacity and add to the safety of the highways.

Roadways of four or more lanes (see fig. 1) are often used, depending upon the volume of traffic. Division of the traffic streams by a medial zone is becoming more and more frequent where four or more traffic lanes are required. The width of the medial zone varies from a few feet to a maximum of 40 feet, depending upon the cost and ease of acquiring the right-of-way, construction costs, and the location and character of the highway. On special roads known as parkways, designed mainly for recreational use, greater widths are used, ranging up to 200 feet.

Materials.—Rigid adherence to high standards has materially raised the general level of the quality of aggregates furnished for concrete. The same is true of improvement in the uniformity of gradation, which has been advanced particularly by the practice of shipping to the construction site coarse aggregates in two or more sizes and recombining them during the process of proportioning. (See fig. 2.) This practice, by reason of the fact that the relation between the amounts of the different sizes may be changed when occasion demands, provides also added opportunity for adjustment of the concrete mixture to obtain the most satisfactory results.

The use of bulk cement has increased and satisfactory methods of handling and batching it have been developed. Its use is in general preferred because of reduced cost and because of the fact that the size of the batch is readily adjusted to the full capacity of the mixer without added labor, as would be necessary in weighing out fractional sacks where sacked cement is used.

There is a growing tendency to demand cements manufactured under restricted specifications to suit certain needs. Much uncertainty exists in regard to the necessity of controlling the composition of portland cement to improve durability, but it is the opinion of many that this property has been overlooked to some degree in the desire to produce high strength. The effect of varying, through wide limits, the composition of cement has been investigated by several agencies, especially with reference to the effect on the resistance of concrete to freezing and thawing. However, no satisfactory answer to this problem has as yet been obtained. There is some inclination to limit the amount of tri-calcium aluminate in an effort to improve durability.

Entirely distinct from disintegration due to frost action is a type of failure which takes the form of abnormal expansion of the concrete and which sometimes does not develop until after several years of service. When this expansion is restrained, multiple cracking followed by ravelling and ultimate disintegration results. There is evidence indicating that this type of failure is due to so-called "delayed unsoundness," probably caused by hard-burned free lime or magnesia in the cement, the effect of which is not revealed in the standard soundness test. To guard against this type of failure, portland cement manufacturers have recently adopted an "autoclave" test in which the constancy of volume of a standard bar of neat cement is observed in an atmosphere of steam at 420° F. for 3 hours.

Proportions.—Because of the high strength of modern portland cements, a much higher concrete strength is now obtainable with a given cement content and water-cement ratio than was formerly possible. As, to insure maximum durability, it is necessary to hold the water-cement ratio within certain limits dependent upon exposure conditions, the application of the water-cement ratio to the design of mixtures is, under some conditions, limited. For instance, under the severe winter conditions of the northern part of the United States it is not considered good practice to use a water-cement ratio by volume exceeding 0.8 even though the strength of the resulting concrete may exceed the design requirements. For this reason the proportion of cement in concrete paving mixtures employed in the northern part of the country is seldom less than 6 sacks per cubic yard and is frequently higher. In the South, where favorable climatic conditions obtain, the cement content is frequently as low as 5.5 sacks per cubic yard and sometimes lower.

Transverse joints.—Early in 1934 the United States Bureau of Public Roads required that expansion and contraction joints should be used in all concrete pavements involving the expenditure of Federal funds. This has stimulated the general use of transverse joints. Previous to that time, many States omitted joints entirely or placed them at long intervals.

The Federal Bureau requires that expansion joints not less than three-fourths inch wide be placed at intervals not exceeding 100 feet in length. In practice the spacing ranges from 40 to 100 feet. The Bureau also requires that contraction joints be placed between the expansion joints so that the transverse joint spacing shall not exceed 30 feet. In practice the transverse joint interval ranges from 10 to 30 feet, generally from 20 to 30 feet. Such spacings are intended to form slab lengths sufficiently short to reduce transverse cracking materially. Only the shortest spacings, however, may be expected to relieve restraint against temperature warping at all points.[4]

Types of joints and joint materials are manifold. The most common types of fillers for expansion joints used previous to 1933 were premolded bituminous felt and poured bituminous materials. Bituminous fillers have the fault of extruding during expansion of the pavement and causing a rough riding surface. Lacking resiliency, they also fail to return to the widening joint when the pavement contracts and so permit infiltration of inert material into the joints which renders them thereafter inoperative for expansion purposes.

Recently various types of nonextruding joint fillers have been used. Among these are cork, rubber, wood or cane fiber, and granular materials such as sawdust and cottonseed hulls. These fillers eliminate the bumps due to extruded material, but they are dependent upon a proper sealing material to prevent infiltration of water and inert material. A recent development is the air-cushion joint which is provided with a seal of thin sheet copper at the top and ends of the joint. These, however, have not been in use for a sufficient length of time to determine their merits definitely. It is recognized that the behavior of the various filler materials cannot be dissociated from the slab lengths with which they are used. Fillers that act poorly with long slabs perform much better when used with short slabs because of smaller movements at the joint.

The contraction joints used are generally of the weakened plane or the dummy types. The weakened-plane type is ordinarily formed by inserting a deformed or straight metal plate in the concrete for the full or part depth of the slab, causing the slab to crack. In some designs the metal plate is provided with a copper seal to prevent the infiltration of water and dirt. The dummy contraction joint usually consists of a groove 1½ to 3 inches in depth formed in the surface of the pavement. The groove is filled with either poured or premolded bituminous filler.

At present it is believed that adequate provision for crack control and proper load transfer across joints will add to the service life of pavements and reduce maintenance costs, and it may be expected that this problem will receive serious study in the future.

Longitudinal joints.—The use of longitudinal joints at lane-width intervals has now become universal in the United States. Their use prevents practically all longitudinal cracking and reduces curling of the slabs. The usual practices employed in the formation of longitudinal joints are: (1) The deep cutting of the surface of the pavement to induce the formation of a crack; or (2) the insertion of deformed metal plates. Tie bars, generally ½-inch rods extending about 1 foot into the pavement at each side of the joint at longitudinal intervals of 2½ feet, serve to hold the sections together and insure load transfer across the joints. In some of the States of the United States each lane is now being built as an independent slab with both edges thickened.

Load transmission-devices.—That a free edge in a pavement is structurally weak has been shown by theoretical analysis, investigation, and experience. Some means of strengthening the edges of slabs at transverse joints is therefore necessary in order to obtain a pavement of balanced design. This is necessary at contraction as well as expansion joints. In some instances this has been accomplished by the use of special reinforcement along the joints, or by thickening the ends of the slabs at the joints in the same manner as the outside edges are thickened. These methods have not in all instances been entirely successful and have never been used extensively in the United States. Load transfer devices, however, which provide mutual support between adjacent slab ends have come into general use during the last few years.

The most common load-transfer device is the dowel bar. These are usually of ¾-inch diameter, 24 inches long, and spaced from 12 to 15 inches along the joints. One-half the length of each dowel is painted and oiled to provide for slippage and a metal cap is placed around the end to form a space in which the dowel slips during movement of the pavement. Other shapes such as T-bars, angles, channels, flat plates, and pipes, have been used in place of dowel bars for expansion joints, because of their greater bearing area and greater stiffness for the same weight of metal. Recently a short dowel bar with close fitting metal sleeve which provides additional bearing area on the concrete has been developed and used quite extensively with apparently good results. There are also many load-transfer devices on the market today. Some are designed as an integral part of the joint, and others can be used with any joint filler.

Recent investigation [5] seems to indicate that doweled transverse joints are effective in relieving stresses caused by expansion, contraction, and warping, but that they are not as effective as might be desired in relieving load stresses near the joint edge. Further investigation and tests are under way in many States which, together with observations of installations in service, should supply data on which definite conclusions can be based.

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[4] PUBLIC ROADS, Vol. 16, No. 10, December 1935.

[5] PUBLIC ROADS, vol. 17, No. 8, October 1936.

Reinforcing.—While there have been no recent outstanding developments in the use of reinforcing steel in concrete pavements, there is an increasing tendency toward the use of moderate amounts of distributed reinforcement; and there is a pronounced feeling that the possibilities of steel reinforcement have not been sufficiently explored. As used in the United States, the reinforcement is not designed to prevent either tensile or flexural cracking. The common practice employs a single layer of well-distributed reinforcement consisting of comparatively small members closely spaced to prevent the opening or widening of cracks. Large members, widely spaced, are not as effective for this purpose. The average usage employs from 50 to 55 pounds of steel per 100 square feet and places it 2 inches below the pavement surface, with members spaced not more than 12 inches apart longitudinally and traversely.

Vibration.—In the placing of structural concrete there has been recently a rapid increase in the use of high-frequency mechanical vibration as an aid toward the production of stronger, denser, and more economical concrete than can be obtained by ordinary hand-placing methods. The progress in applying the same principle in the placing of pavement concrete has been slower, possibly because of the difficulty of designing vibratory equipment suitable for the purpose.

The United States Bureau of Public Roads and several States, by constructing experimental sections of pavement with vibratory equipment, have contributed most of the information that is available on the subject. Preliminary investigations, in which the proportions of materials, consistency of the mixture, and quality of the resulting concrete were given a thorough study, have been followed by investigations on regular contract sections to study the practicability of the method for general admission under specifications for concrete pavement construction. The results obtained show, in general, considerable advantage of the vibratory methods over the conventional methods.

It has been fairly well established that concrete, unworkable by conventional means of placing, follows the same laws as concrete of consistencies employed with conventional finishing methods, when it is made plastic by vibration during the process of placing. In other words, vibration imparts no new property to the concrete and the water-cement and voids-cement ratio laws still hold good.

From the standpoint of economy the advantages of vibration are established without doubt. Dry and harsh mixtures of from ½ to 1-inch slump, formed either by reducing the water-cement ratio or by using additional aggregate above that used with conventional methods, have been satisfactorily placed by use of the vibratory method. In either case it has been necessary to reduce the proportion of fine to coarse aggregate in order to obtain mixtures suitable for use with the vibratory equipment. The increase in strength, obtained at no increase in cost of materials by lowering the water-cement ratio, has amounted to about 10 percent.[6] Reduction in the cost of the concrete, by saving from 10 to 15 percent of the cement, has been obtained without sacrifice in strength by increasing the amounts of the aggregates and maintaining the water-cement ratio employed with the conventional method. In addition, the vibrated concrete has been found to be more dense, less absorptive, and less honeycombed than that placed by conventional methods.

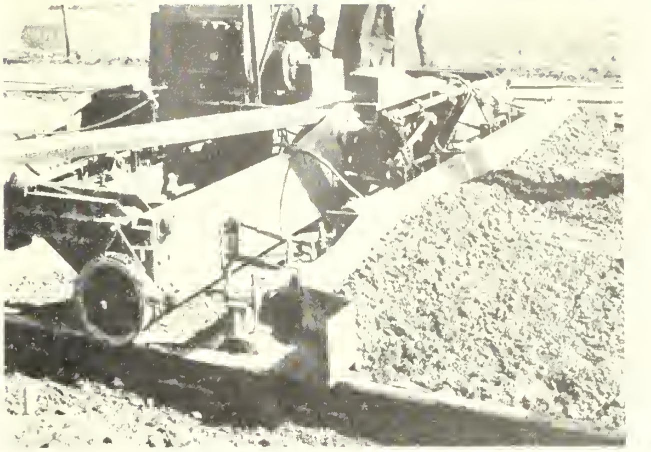

Figure 3.—Finishing Machine with Vibrators Mounted on the Front Screed.

The vibratory equipment used on regular contract sections has proved entirely practical. No mechanical difficulty of any consequence has been encountered. No increase in construction personnel has been necessary, because the additional labor required to spread the concrete has been offset by the elimination of the necessity of spading adjacent to the forms. No additional equipment has been necessary. The concrete has been consolidated satisfactorily around the transverse joints without damaging them, and the rate of progress has been at least as satisfactory as when conventional equipment and methods were used.

The principles of both surface and internal vibration have been applied to placing concrete pavements, although up to the present time the former perhaps has been most used. The frequency of vibration in most instances is 3,600 impulses per minute or slightly more.

Surface vibration has been accomplished by mounting vibrator units on the front screed of the finishing machine (see fig. 3) or by mounting them on a pan between the screeds and independent of the screeds. Both methods have given very good results. Another method adapts the principle of internal vibration by mounting vibrator units on a pipe suspended in the concrete ahead of the finishing machine. By still another, the vibratory action is applied through a shaft, with protruding pins, which is made to travel on the forms ahead of the finishing machine. These two latter methods are relatively untried at the present time. Other methods, such as an arrangement by which the concrete is vibrated as it passes through the throat of a hopper from which it is spread on the subgrade, are in the development stage.

It is felt that the maximum advantages that may be had through the use of vibration have not yet been obtained. It should be possible to place and finish still drier and harsher mixtures and obtain concrete of still greater density and durability. To achieve this end it will be necessary to redesign other units of the production equipment to handle the material. This

(*Continued on page 71*)

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[6] Public Roads, Vol. 18, No. 2, April 1937.

DEVELOPMENTS IN BRICK PAVEMENTS IN THE UNITED STATES

Reported by JOHN JASTER, Jr., Director, Ohio State Highway Department, Principal Reporter; GEORGE F. SCHLESINGER, Engineer Director, National Paving Brick Association; JOHN S. CRANDELL, Professor of Highway Engineering, University of Illinois; and ROY L. PHILLIPS, City Engineer, Meadville, Pa.[1]

INTRODUCTION

THE brick pavement structure as constructed in the United States, consists essentially of the three following component parts. The surface course is composed of rectangular vitrified clay units assembled in regular pattern with the joints or interstices completely filled and sealed, binding the units together. The bed or cushion course, is a thin layer of granular material on which the paving brick units are laid and embedded and which will permit slight adjustments necessitated by irregularities in the foundation or by permissible variations in the depths of the brick. The base course or foundation, supports the bed and surface and distributes traffic loads to the subgrade.

PAVING BRICK UNIT

Manufacture.—Paving brick are formed by what is known as the "stiff-mud" process of clay products manufacture. The shale or clay after being ground very fine is thoroughly mixed with water to the proper consistency. A powerful auger then forces the stiff mixture through a die on to a moving belt in the form of a column. This column is carried to the cutter where taut wires automatically cut it into brick units. In so-called vertical fiber paving brick the width and length are determined by the dimensions of the die and the depth by the spacing of the wires on the cutter. In the wire-cut-lug type the depth is the shorter dimension of the die and the lugs are formed by an eccentric motion given to the cutting wires. In this type the smooth or die side of the brick is in the surface of the pavement. When the brick are re-pressed before burning the surface is also smooth.

Formerly the re-pressed and the wire-cut-lug types were in general use and their preference was in a large degree based on their having side lugs considered essential for the proper filling and sealing of the joints. However, the manufacture of brick having a wire-cut wearing surface with lugs on the side and ends is now commercially practical and this type, known as vertical fiber lug, is now the most popular single variety in use in the United States. Regardless of the type of vitrified paving unit that may result from future developments it will undoubtedly have a roughened or deformed wearing surface of nonskid character.

De-aired brick.—Most paving brick plants are equipped with machinery for evacuating the air from the mixed clay or shale during the process of manufacture. De-airing with proper methods will produce a brick of greater strength and density and improved regularity of shape. Glassy structure causing brittleness must be guarded against and accurate control in manufacture is necessary in order to obtain the full advantages of the process. The Research Bureau of the National Paving Brick Association at the Ohio State University Experimental Station at Columbus, Ohio, has made observations on the internal structure of paving bricks manufactured with various degrees of vacuum and methods of forming with a view to the further perfecting of plant practices. In some cases authorities have specified that the paving brick be de-aired although usually the physical requirements govern with no mention of the method of manufacture. In the brick to be used for paving the first tube of the new Lincoln Tunnel under the Hudson River in New York City, a maximum of 18 percent rattler loss for 3 by 4 by 8½-inch de-aired brick is the requirement.

Standard types and sizes.—Since 1921 the Division of Simplified Practice of the United States Bureau of Standards, through a committee composed of representatives of engineering and technical societies, has annually recommended a standard list of sizes and varieties of paving brick. The current recognized list is given in table 1.

TABLE 1.—*Standard list of sizes and varieties of paving brick*

Variety	Size			1936 shipments
	Depth	Width	Length	
	Inches	*Inches*	*Inches*	*Percent*
Repressed Lug	4	3½	8½	7.1
Vertical fiber lug	2½	4	8½	12.8
Do	3	4	8½	51.3
Do	3½	4	8½	9.3
Total				80.5

It will be noted that the list does not contain lugless varieties which at one time were in general use in the western part of the United States. The proportion of vertical fiber lug brick shipped in 1936 (73.4 percent) is the greatest for any type since standardization was initiated.

Specifications.—Practically all standard specifications for the physical qualities of paving brick used in the United States are identical with, or in essential details are based on those of the American Society for Testing Materials.[2] The requirements in regard to the abrasive loss in the rattler test are sometimes varied (usually downward from the A. S. T. M. requirements) to apply to the quality of brick available in a particular locality. The standard specification for paving brick of the A. S. T. M. was revised in 1937. The most notable innovation in the new specification is the additional requirement to the rattler test limiting the number of broken pieces weighing 1 pound or more.

Some specifications, notably those of the Highway Department of the State of Ohio which is a large user of paving brick, had in recent years added a test for flexural strength. An investigation indicated that there was close coordination between broken brick in the rattler and low flexural strength. It is believed that a broken brick limitation will be more determinate than the flexural test in that it will detect brick that have minute cracks on only one surface. In addition it is considered a criterion of toughness. This is the first time that the rattler test, which was devised in the United States, has been modified, although it is realized that its application by stages is a part of paving brick testing in Holland.

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[1] The following information was submitted by the reporters listed and organized into one report by the principal reporter. Information on Manufacture, Specifications, Construction Methods, Fillers, and Base Courses was furnished by Mr. Schlesinger. The section on Bed Courses was written by Professor Crandell. Mr. Phillips submitted the information on Municipal Use.

[2] American Society of Testing Materials Designation C7—37 T.

CONSTRUCTION METHODS

While the type and quality of the paving brick unit is a matter of fundamental importance, its method of utilization is at least of equal moment. Brick in the pavement are surrounded and supported by a number of other materials of construction. The excellence of the pavement structure as a whole is the objective of design and construction requirements. Most of the researches and developments concern improvements in what may be termed "auxiliary materials" and the manner in which they are used.

Bed course.—For many years the sand cushion has been used throughout the world as the standard type of bed course on which to lay paving brick. In 1920 a new type was tried at Syracuse, N. Y. It consisted of a mixture of light refined tar and clean, sharp sand in the proportion of 7 percent coal tar and 93 percent sand by weight. These materials were mixed together and laid cold. The pavement has been examined from time to time since then and it is in excellent condition today, showing no signs of movement of the cushion.

Since that time this bituminous-bound type of cushion or mastic bed has found favor in many parts of the United States, and is now the standard in several States and cities. The specifications for mastic bed of the American Public Works Association (formerly the American Society of Municipal Engineers[3]) are typical of those in most general use. In most cases a liquid asphalt has been substituted for coal tar since the asphaltic product is cheaper. If the correct proportion of asphalt is used the results seem to be as good as those that are obtained with coal tar, but too often the contractor has been allowed to reduce the amount of asphalt to as low as 3½ percent, because he has claimed that it is difficult to work with a mixture as "fat" as was originally called for. This is sheer nonsense, and the cushions laid with the leaner mixes soon show signs of deterioration and eventual failure.

Water-gas tar has been used as a binder to some extent, but the results have not been generally satisfactory, as evidenced by the Chestnut Street pavement, Philadelphia, Pa. Some of the cut-back coal tar products have not given good results either, and their use is not recommended. Asphaltic emulsions are not satisfactory. Best results have been obtained with light, refined straight-run coal tars, as well as those asphaltic cut-backs which do not cure too rapidly.

The sand used for bituminous cushions is important. It should be clean and sharp. A rounded sand, such as is often to be found in river bottoms, or along swift running streams, is, in general, unsatisfactory, because the resulting cushion is not stable. It tends to be rolled up into the joints when the power-roller passes over the brick. A sharp sand resists this tendency, and the cushion made with it stays where it is deposited.

More recently experiment has been made with rock asphalt as a cushion course. A lean rock asphalt has been tried, containing about 3 percent of asphalt. It seems to have some points of superiority, such as ease of handling, and a tendency to set up and resist movement under the wearing surface. It is too early, however, to present a full report on this as the sections observed in this country have been in use less than 2 years.

In the early practice the bedding course or cushion varied in thickness to accommodate irregularities in the roughly finished base. This frequently resulted in shifting of the cushion with consequent effect on the brick surface. Present design calls for a much thinner bed with a uniform thickness not greater than three-fourth inch, which is possible because of a correspondingly smooth base-course surface requirement.

Fillers.—The surface-removal method of bituminous filler application introduced several years ago has now become practically universal practice in the United States. Immediately prior to the application of the filler, the brick are treated with a separating agent of lime and water, or calcium chloride, laundry starch, and water. These agents are sprayed on the surface of the brick in place in the pavement in such way as to coat the exposed surface of the brick without penetrating the joints. When the separating coating has been thus applied the filler is poured directly into the brick joints and the surplus scraped off the top so as to leave the surface of the pavement clean and the joints full.

Investigations reported to the Highway Research Board[4] in 1933 and 1934 indicated that the coefficient of friction, both rolling and sliding, on "a vertical fiber brick road, free of asphalt filler" was relatively high. Observations made on the same pavement during the second and third years after completion indicated a measurable reduction in friction. According to these reports, this was caused by the asphalt progressively exuding from the joints in hot weather and covering a considerable percentage of the surface.

A prominent part of the work of the Research Bureau of the National Paving Brick Association has been concerned with the development of paving brick joint fillers that will not exude in hot weather. Consideration has been given to cement grouts, bituminized cement grouts, plasticized sulphurs, and bituminous fillers. Among the bituminous fillers, comparisons have been made between asphalts from different base crudes, asphalts of different softening points and penetrations, asphalt mastics, and straight pitches and pitch mastics. In the laboratory the properties and behaviors of the fillers were observed and interpreted in terms of practicability. Special emphasis was given to a test wherein the exuding or receding tendencies of the bituminous fillers were observed by subjecting filled brick panels (about 1 square yard in area) to prolonged periods of simulated summer temperatures. From these tests a number of fillers were selected as worthy of actual pavement trial.

In cooperation with the Ohio Department of Highways and the United States Bureau of Public Roads, a project was planned and completed (November 1935) in which the entire length of 1¼ miles of new brick pavement on route 31 in Hocking County, Ohio, was allotted to a test of fillers. Of these fillers 13 are in sections exceeding 300 feet in length and 8 in sections somewhat shorter. During construction, observations were made to determine the practicability of application, including the surface removal of the fillers. While this pavement has been in service but little more than 2 years, the weather experienced has been exceptionally severe. During the winters there have been prolonged periods of subzero weather. The summers, of exceptional warmth, included periods during which temperatures exceeded 100° F.

The Ohio Department of Highways, basing its selection on satisfactory behavior in the test road to date has designated four of the fillers for more extended use and further trial in brick pavement projects. In 1937 these 4 fillers were optional requirements on 12 projects. They can be described in general terms as follows:

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[3] American Society of Municipal Engineers, Standard Specification, adopted October 1935.

[4] Proceedings of the Highway Research Board, December 1933, p. 169 and December 1934, p. 131.

1. A "blended" asphalt, 65 percent midcontinent base and 35 percent asphaltic base; penetration at 25° C., 23 to 32; softening point 101°–110° C.
2. A blended asphalt similar to the above, with 20 to 30 percent finely divided mineral content; penetration at 25° C., 17 to 26; softening point 107°–116° C.
3. A special coal-tar pitch; penetration at 25° C., 35 to 65; softening point 60°–75° C.
4. A plasticized sulphur-asphalt mixture, with sulphur content 38 to 42 percent; penetration at 25° C., 28–34; softening point 65°–75° C.

Typical 3-Lane Brick Pavement on U S 30 in Ohio.

With the removal of the excess mat on top, a more thorough inspection of the pavement is possible. One consequence has been the requiring of end lugs instead of beveled ends (bulged). These lugs on each end are non-meshing, so as to provide for free flow of filler, and function equally well when the brick are turned.

Base courses. The requirements for foundations or base courses vary according to conditions of climate, subsoil, and traffic loads. In the southern section of the United States, vitrified brick surface courses are successfully used on such foundations as natural sand, gravel, crushed stone, slag, shell, and Florida lime rock. Brick pavements have, of course, given excellent service on bases of waterbound and bituminous macadam, of bituminous concrete, and of portland-cement concrete.

Concrete is the material commonly used as a foundation course for city streets and heavy-traffic pavements. With proper subgrade conditions most engineers prefer a relatively lean concrete for the base. It will be less affected by temperature changes and, because of its lower tensile strength, the cracks that form are more numerous but narrower. The section of the base, separated by such narrow cracks, are displaced relatively one to another with greater difficulty than if the cracks were wider as would be the case if the concrete were stronger; and hence are less likely to affect the surface course. Reinforcing with steel mesh or bar mats, usually in lieu of a richer concrete, is an increasing practice. Expansion joints in the base course are a cause of disturbance of the brick surface and are not recommended. With plain concrete, transverse weakened plane or construction joints are used to a considerable extent. As in concrete pavements, dowelled longitudinal joints should separate the base into sections of 1-lane width. With integral curbs, which perform the same function, thickened edges are not required.

In connection with the discussion of joints in concrete bases it is apropos to mention that the monolithic type of brick pavement shows some signs of revival. In this design the brick are laid, without cushion, directly on the green concrete base and filled with cement grout. In an experimental brick pavement on route No. 43 in Carroll County, Ohio, constructed in 1933, a section of monolithic pavement was included. The construction included a longitudinal center joint and 1-inch transverse expansion joints at intervals varying from 50 to 100 feet. A smooth surface was formed and the section is now in perfect condition. Further projects of this design are contemplated. It would seem certain, however, that even if the monolithic type should again receive favor, the present standard brick pavement with cushion and flexible filler, which has proved its merits, will continue to be the preferred type for general use, particularly for municipal streets and resurfacing.

Longitudinal laying.—In December 1936 the Ohio Department of Highways completed with Federal aid the construction of a section of brick pavement on route 31 (located a short distance from the filler test road previously mentioned) in which the brick were laid longitudinally—that is with the 8½-inch dimension parallel with the curbs. Longitudinal laying was not a complete innovation as it had been used previously for car-track areas, parking strips, on grades, and, to a limited extent, in normal installations. However, this project was the first of its kind using modern construction methods. Contemplated advantages included smoother riding, reduction in traffic noise, spanning of base cracks lengthwise, a reduction in breaking of brick for transverse closures, economy in labor of laying because of unlimited length of rows, and the elimination of special wedge-shaped sections on curves. The project, completed in 1936, was 1.35 miles long, 20 feet wide over-all, had a concrete base, with 9-inch integral curbs, and used 3-inch vertical-fiber lug brick and standard asphalt filler.

An extension of this project for 1.5 miles using nonexuding filler was under construction in September 1937. The specifications for the first project did not permit "batting" but it was necessary at infrequent intervals to cut brick at transition points. Also because brick were respaced or shifted on the cushion, the resulting surface contour was not as smooth as expected. On the second project, batting will be permitted at one curb and it is believed that the appearance of the pavement will not be noticeably affected, and that economy in laying and a smoother surface will result.

Continuous longitudinal joints would be of evident disadvantage if traffic were predominantly steel-tired. However, this type of traffic is practically nonexistent in the United States. The future performance of these projects in service will determine whether the apparent advantages of longitudinal laying are sufficient to justify a change in present standard practice.

Reinforced brick pavement.—Reinforced brick masonry in which steel rods are embedded in the mortar joints has been used to a considerable extent in recent years in pavement construction. This is particularly true in California where it is considered desirable because of earthquake conditions. In June 1931 a reinforced brick

(*Continued on page 72*)

PROGRESS IN THE PREPARATION AND USE ON ROADS OF TAR, ASPHALT, AND EMULSIONS

By E. F. KELLEY, Chief, Division of Tests, United States Bureau of Public Roads, Principal Reporter

INTRODUCTION

NO REPORT on this subject has been made by the United States to the Permanent International Association of Road Congresses since the Congress of Washington. Therefore this report will not be limited in scope to the progress that has been made since the Congress of Munich but will be extended to include the more important developments that have taken place since 1930.

In order to avoid the possibility of misunderstanding it seems necessary in this report, as in that of 1930, to define the accepted meaning in the United States of some of the terms relating to the materials that are the subject of discussion. This is necessary because certain terms do not have the same meaning in the United States that they have in other countries.

As used in the United States and in this report the significance of the terms that appear to require definition are as follows:

Bitumen is that portion of tar, petroleum, asphalt, or asphaltic material, that is completely soluble in carbon disulphide.

The adjective "bituminous" means "containing bitumen." Thus, tars, asphalts, and asphaltic products are all bituminous materials. Therefore the terms "bituminous mixture," "bituminous treatment," "bituminous construction" are general ones and indicate only that the use of either tar or asphaltic material is involved.

Tar is a bituminous product which yields pitch when partially evaporated or fractionally distilled and is produced by the destructive distillation of organic materials such as coal, petroleum, wood, lignite, and peat. The tars used in the United States for road purposes are manufactured almost entirely from coal tar and water-gas tar. A very limited amount of wood tar has been used experimentally.

Asphalt is a solid and semisolid cementitious material, ranging in color from black to dark brown, in which the predominating constituent is bitumen. Asphalts may be those which occur in nature or those produced by refining petroleum.

Since asphalt is defined as a solid or semisolid material it is necessary to differentiate between it and those materials that contain asphalt but are liquid in character. Prior to 1930 the term "road oil" was applied rather indiscriminately to liquid asphaltic materials used for road purposes with a great deal of resultant confusion. Since then the use of that term has been discouraged and the liquid products are now designated as "liquid asphaltic road materials."

Cut-back asphalts are liquid materials produced by fluxing solid or semisolid asphalts with volatile petroleum distillates.

The liquid asphaltic road materials are divided into three general classes depending on the degree of hardening or cementitiousness that is desired and the rapidity with which it is developed. These classes are designated as slow-curing, medium-curing, and rapid-curing. The slow-curing materials are intended to harden very slowly after use and to develop only relatively low cementing values. The medium- and rapid-curing materials are cut-back asphalts and, in contrast to the slow-curing materials, are intended to harden at a relatively rapid rate and, through loss of the volatile solvents, to leave in place a residue of semisolid asphalt. The solvents used in the manufacture of cut-back asphalts are of the kerosene type for medium-curing materials and of the gasoline or naphtha types for the rapid-curing materials.

USAGE OF BITUMINOUS ROAD MATERIALS

The past few years have seen a great increase in the use of bituminous road materials in the United States, particularly in the use of the liquid products.

In 1929 the total consumption of asphaltic road materials was 2,656,000 tons of which the slow-curing liquid products and cut-back asphalts accounted for only 35 percent. In 1936 these liquid materials constituted 63 percent of a total consumption of 3,807,000 tons.[1]

With respect to road tars it is estimated that the annual consumption during this same period increased from more than 100,000,000 gallons to more than 150,000,000 gallons.[2]

Tar emulsions are not used in the United States and no reliable figures are available concerning the consumption of asphaltic emulsions in 1929 or 1930. However, their use at that time was relatively small, while in 1936 the estimated consumption was 56,000,000 gallons.[1]

SPECIFICATIONS

No changes of importance have been made in the specifications commonly used for the semisolid asphalts known as asphalt cements or paving asphalts. However, the increased use of liquid bituminous materials has resulted in the development of a number of new and more adequate specifications for these products.

In 1932 the United States Bureau of Public Roads cooperated with the Asphalt Institute, representing the asphalt producers, in formulating a group of specifications for liquid asphaltic road materials of the slow-curing, medium-curing, and rapid-curing types. The requirements of these specifications are given in table 1. These specifications were recommended to the State highway departments for adoption and, while the States as a whole have not seen fit to adopt them in their entirety, they have had a marked influence on the specifications now in use. In 1933 the requirements of the recommended specifications for the medium-curing and rapid-curing cut-back asphalts were approved for inclusion in the specifications of the Federal Government.

The improvement and standardization of specifications for tar have been aided considerably by the adoption in 1937, by the American Association of State Highway Officials, of the Standard Specifications for Tar for Use in Road Construction, as given in table 2. These specifications have been adopted in whole or in part by several States and their adoption, possibly with slight modifications, is now being considered by the American Society for Testing Materials and the Federal Government. The important improvement in these specifications, as compared with preceding ones,

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[1] Figures given are from published statistics of the United States Bureau of Mines.

[2] Unofficial estimates; no official figures available.

TABLE 1.—*Specifications for liquid asphaltic road materials recommended in 1932 by the U. S. Bureau of Public Roads and the Asphalt Institute*

Test requirements [1]	Grades [2]												
	SC-1	SC-2	SC-3	SC-4	MC-1	MC-2	MC-3	MC-4	MC-5	RC-1	RC-2	RC-3	RC-4
Water and sediment, percent	2.0−	2.0−	2.0−	2.0−									
Flash point, °F	150+	200+	200+	250+		150+	150+	150+	150+	80+	80+	80+	80+
Viscosity, Saybolt-Furol, seconds:													
At 77° F	20–150				40–150								
At 122° F		200–320								80–160	200–400		
At 140° F			150–300	350–550		150–250	300–500	500–800				275–400	700–1,400
At 180° F									170–280				
Total distillate, percent by volume:													
To 374° F										5+			
To 437° F		2−	2	2−	10−	2−	2−	1−	1−	12+	10+	3+	0.5+
To 600° F		15−	10−	8−	25+	10–20	8–20	16−	14−	25+	20+	14+	7+
To 680° F	50	25−	20−	18−	50−	27−	25−	25−	20−	40−	35−	30−	25−
Float of residue, 122° F., seconds	50−	25+	25+	25+									
Penetration of residue, 100 g, 5 seconds, 77° F					70–300	100–300	100–300	100–300	100–300	60–120	60–120	60–120	60–120
Ductility of residue, 77° F., cm					60+	60+	60+	60+	60+	60+	60+	60+	60+
Solubility of residue in CS_2, percent	99.0+	99.0+	99.0+	99.0+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+
TYPICAL USES [3]													
Dust layer	X												
Primer					X								
Surface treatments										X			
Mixed-in-place construction:													
Dense-graded aggregate [4]		X	[5] X			X							
Dense-graded aggregate [6]											X	[5] X	
Open-graded aggregate [7]												X	
Plant-mixed construction:													
Dense-graded aggregate [4]			[5] X	X			X						
Dense-graded aggregate [6]								X	X		X	X	
Open-graded aggregate [7]												X	X

[1] Methods of test are those of the American Association of State Highway Officials.
[2] + indicates that the value shown is the minimum allowable; − indicates that the value shown is the maximum allowable.
[3] For the guidance of the user and not a part of the specification.
[4] Maximum size not over 1 inch. Fairly uniform grading from coarse to fine with appreciable percentage passing No. 200 sieve.
[5] Under favorable conditions only.
[6] Maximum size not over 1 inch. Fairly uniform grading from coarse to fine with little or no material passing No. 200 sieve.
[7] Maximum size not over 1½ inches. Little or no material passing No. 4 sieve.

is in the establishment of 12 numbered grades through which the consistency increases from minimum to maximum in a logical manner. That this is true is not readily apparent except when the consistencies of the different grades are measured at a single temperature and expressed in terms of absolute viscosity. The consistency requirements of these specifications are based on absolute viscosity as measured with a capillary rise viscosimeter that has been developed recently (*1*).[3] However, since few laboratories are now equipped with instruments for the measurement of absolute viscosity, the consistency requirements of the specifications are still expressed in terms of specific viscosity, Engler, and the float test.

Much interest is being shown in the proposal to specify viscosity requirements in terms of absolute units, both for tars and for asphaltic materials, and a number of highway laboratories are investigating its possibilities.

Three distinct types of emulsified asphalt are in use in the United States. These types may be classified as follows:

1. Quick-breaking emulsions for penetrations (or grouting) where there is no manipulation of the aggregate after it has been coated.
2. Medium-breaking emulsions for mixing with clean, coarse aggregate.
3. Slow-breaking emulsions for use in densely graded mixtures containing stone dust, sand, or other fine mineral aggregate. Emulsions for soil-stabilization belong to this general class.

The American Association of State Highway Officials has adopted standard specifications for five grades of emulsified asphalt, the requirements of which are given in table 3. Specification M–51 covers a quick-breaking emulsion suitable for penetration macadam and for surface treatments. Specifications M–47 and M–48 are for medium-breaking emulsions for coarse aggregate mixtures of the plant-mixed and mixed-in-place types, respectively. Specifications M–49 and M–50 are for heavy emulsions to be used in summer and winter, respectively, in the preparation of bituminous concrete for repair work. The American Society for Testing Materials had adopted tentative standard specifications for four grades of emulsified asphalt practically identical with the first four grades shown in table 3.

Several of the tests used to measure the desired qualities of emulsified asphalts are of quite recent origin and merit brief mention.

The demulsibility test is for the purpose of insuring that the emulsion will break at the desired rate. In this test an electrolyte ($CaCl_2$) in weak solution is added to 100 grams of emulsified asphalt and the percentage of the asphalt that is coalesced by contact with the electrolyte is determined. For quick-breaking emulsions a high percentage of coalescence in a weak solution of $CaCl_2$ is required, and for slower-breaking emulsions a low percentage of coalescence in a more concentrated solution.

The settlement test is a sedimentation test to determine if the distribution of the asphalt in the liquid phase will remain reasonably uniform over an extended period of time. Five hundred milliliters of emulsion are placed in a stoppered glass cylinder 5 centimeters

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[3] Italic figures in parentheses refer to the bibliography, p. 70.

Table 2.—*American Association of State Highway Officials Standard specification for tar for use in road construction. Specification M 52; adopted 1937*

Test requirements [1]	Grades [2]													
	RT-1	RT-2	RT-3	RT-4	RT-5	RT-6	RT-7	RT-8	RT 9	RT-10	RT-11	RT-12	RTCB-5	RTCB-6
Consistency:														
Specific viscosity, Engler, at 40° C	5-8	8 13	13 22	22-35										
Specific viscosity, Engler, at 50° C					17-26	26-40							17-26	26-40
Float test at 32° C							50-80	80-120	120-200					
Float test at 50° C										75-100	100-150	150-220		
Specific gravity at 25° C./25° C	1.08+	1.08+	1.09+	1.09+	1.10+	1.10+	1.12+	1.14+	1.14+	1.15+	1.16+	1.16+	1.09+	1.09+
Total bitumen, percent by weight	88+	88+	88+	88+	83+	83+	78+	78+	78+	75+	75+	75+	80+	80+
Water, percent by volume	2.0−	2.0−	2.0−	2.0−	1.5−	1.5−	1.0−	.0	.0	.0	.0	.0	1 0−	1.0−
Total distillates, percent by weight:														
To 170° C	7.0−	7.0−	7.0−	5 0−	5.0	5.0−	3.0−	1.0−	1.0−	1.0−	1.0−	1.0−	[3]1.0-8.0	[3]1.0 8.0
To 200° C													5 0+	5.0+
To 235° C													8.0-18.0	8.0-18.0
To 270° C	35.0−	35.0−	30.0−	30.0−	25 0	25.0−	20.0−	15.0−	15 0−	10.0−	10 0−	10.0−		
To 300° C	45 0−	45.0	40.0−	40.0−	35.0−	35.0−	30.0−	25.0−	25.0−	20.0−	20.0−	20.0	35.0−	35.0−
Softening point of residue, ° C	35-60	35-60	35-60	35-60	35-65	35-65	35-65	35-65	35-65	40-70	40-70	40 70	40-70	40 70
Typical uses and suggested temperatures for application.[4]	Prime coat; 60° to 125° F.		Prime coat and surface treatment; 80° to 150° F.		Surface treatment and road mix; 80° to 150° F.		Surface treatment, road mix, premix and seal coat; 150° to 225° F.			Surface treatment premix, seal coat, penetration and crack filler; 175° to 250° F.			Surface treatment road mix and premix when low temperature application and quick setting are desired; 60° to 120° F.	

[1] Methods of test are those of the American Association of State Highway Officials.
[2] + indicates that value shown is the minimum allowable; − indicates that value shown is the maximum allowable.
[3] This requirement was revised in 1938 to 2.0-8.0.
[4] For the guidance of the user and not a part of the specification.

Table 3.—*American Association of State Highway Official standard specifications for emulsified asphalt, adopted 1937*

Test requirements [1]	Specification No.[2]				
	M 47	M-48	M-51	M 49	M-50
Viscosity, Saybolt-Furol, 60 ml at 77° F., seconds	100+	20-100	20-100		
Residue by distillation, percent by weight	60-65	55-60	55-60	65+	65+.
Settlement, 5 days, percent	5−	5−	3−		
Demulsibility, percent by weight:					
50 ml of 0.1 N $CaCl_2$	[3] 30−	[3] 30−			
35 ml of 0.02 N $CaCl_2$			60+		
Sieve test, percent by weight	0.10−	0.10−	0.10−	0.10−	0.10−.
Miscibility with water, no appreciable coagulation in 2 hours	(4)	(4)		(4)	(4).
Coating test—thorough coating and no appreciable separation of emulsion	(4)	(4)		(4)	(4).
Freezing test—homogeneous after 3 cycles					(4).
Residue from distillation:					
Penetration, 100 g, 5 seconds, 77° F	100-200	100-200	100-200	100-200	125-225.
Specific gravity at 77° F	1.00+	1.00+	1.00+	1.00+	1.00+.
Ductility at 77° F., cm	40+	40+	40+	40+	40+.
Solubility in carbon disulphide, percent	95+	95+	95+	95+	95+.
Ash, percent	2 0	2.0−	2.0−	2.0−	2 0−
Specified uses	Open-graded course aggregate plant mixtures.	Open-graded course aggregate mixed-in-place construction.	Penetration macadam and surface treatment.	Bituminous concrete mixtures for repair work, summer use.	Bituminous concrete mixtures for repair work, winter use.

[1] Methods of test are those of the American Association of State Highway Officials.
[2] + indicates that the value shown is the minimum allowable; − indicates that the value shown is the maximum allowable.
[3] Specifications M-47 and M-48 were revised in 1938 by the addition of the following footnote applicable to the demulsibility test:
The limit shown is for general use. In case it is desired to specify more restrictive requirements based on prevailing climatic conditions, the following limits are suggested: For hot, dry climates, 0-5 percent; for mild climates, 5-30 percent; for cool, damp climates, 20-45 percent.
[4] Required.

in diameter and allowed to stand for 5 days. At the end of that time the top 10 percent and the bottom 10 percent of the emulsion in the cylinder are removed and a determination made of the numerical difference between the percentages of asphalt in each.

In the sieve test the emulsion is passed through a No. 20 sieve, previously wetted with a 2 percent solution of sodium oleate, and is washed through the sieve with the oleate solution. The residue on the sieve is weighed to determine the amount of coagulated asphalt in the emulsion

The test for miscibility with water is merely to insure that the emulsion can be diluted with water prior to use. The emulsion is mixed with distilled water to determine if coagulation takes place. The test is not applicable to quick-setting emulsions.

In the coating test the emulsion is mixed for 3 minutes with a standard reference stone ranging in size from one-fourth to three-fourths inch. The test is to determine if the emulsion will coat the stone thoroughly and if the mixing operation will cause appreciable separation of the asphalt from the water of the emulsion.

The freezing test, applicable only to emulsions for use in cold weather, is to determine if the emulsion can be subjected to low temperatures without detriment. The emulsion is subjected to alternate cycles of freezing and thawing to determine if it will remain homogeneous.

The other tests listed in table 3 are the same as those applied to other asphaltic materials and require no description.

The slow-breaking mixing emulsions are being used to a considerable extent in densely graded bituminous mixtures but, as yet, there are no standard specifications for them. In general, the requirements for these materials are similar to those for emulsions of the medium-breaking type except for demulsibility and the addition of a mixing test to determine if the emulsion can be mixed successfully with fine-grained material. The test for demulsibility is the same as that for medium-breaking emulsions but the amount of breakdown is usually limited to a maximum of about 2 percent. In the mixing test that is frequently used, the emulsion is mixed with portland cement and water, the amount of breakdown permitted being limited to a low percentage.

Emulsified asphalt for soil stabilization is a special grade differing from the others in that it must be capable of being mixed with large quantities of extremely fine soil materials of the clay size and it should dry rapidly after being mixed. The mixing test with portland cement and a dehydration test are frequently used for this grade of emulsion to insure that it will have the desired properties. Aside from the dehydration test, which measures the rate at which the emulsion loses water, the requirements for stabilization of emulsions are much the same as for slow-breaking mixing emulsions except that a harder grade of asphalt (50 to 100 penetration) is generally used.

The foregoing discussion of emulsified asphalts has related entirely to emulsions of asphalt in water. In addition to the extensive use of these, there has been a limited use in the United States of so-called inverted emulsions or emulsions of water in asphalt. However, their use has not been widespread and no effort has been made, as yet, to effect any standardization of requirements designed to insure their satisfactory quality.

In addition to the developments in specifications for the bituminous materials themselves, an important change has taken place in specification requirements for the coarse mineral aggregates with which they are used. This has been the rather general adoption of the Los Angeles abrasion test to replace the Page toughness test and the Deval abrasion test that have been standard for many years.

The latter two tests have a number of defects among which the following are worthy of note. The Page toughness test is made on samples taken from ledge rock and therefore is not applicable to crushed slag or gravel. The Deval abrasion test, as applied to stone, is also made on samples from ledge rock and therefore it has been necessary to modify the test method when it is used for gravel or crushed slag and it has also been necessary to specify different maximum losses for these materials than have been specified for stone. Finally, and most important, the results of these tests are not in good agreement with the observed performance of the aggregates as used in road surfaces. The Los Angeles abrasion test is not subject to the criticism that has been directed at the Page toughness test and the Deval abrasion test. It is applicable alike to crushed stone, crushed or rounded gravel, or crushed slag, and these materials may be tested after being crushed and screened to the sizes that are to be used. Moreover, there is a good correlation between test results and the breakage and crushing of the coarse aggregates that takes place under construction rollers and under traffic.

TYPES OF CONSTRUCTION

Hot-mixed pavements.—The principal types of hot-mixed and hot-laid bituminous pavements used in the United States are coarse-aggregate asphaltic concrete, fine-aggregate asphaltic concrete, and sheet asphalt. It is unnecessary to describe them in detail here since this has been done in the report of 1930 and in other literature. No essential changes have been made in recent years in the design and construction of these types, except in the increased use of improved construction equipment. The new equipment will be described later.

In addition to these standard types that are mixed hot and laid hot, a number of proprietary mixtures have been developed that are designed to be laid cold. These differ from the standard mixtures primarily in the kind of asphaltic materials, or the combinations of them, that are used and in the special procedures that are employed in their preparation.

The heavier grades of tar (RT–8 to RT–12, table 2) are also used to a limited extent in dense graded pavements that are mixed hot and laid, either hot or cold, in two courses. The principal difference between those designed to be laid hot and those designed to be laid cold is that the former have greater density and greater initial stability due to the use of finer aggregates and heavier tar cement.

Bituminous macadam.—The construction of bituminous penetration macadam with asphalt cement or tar does not require description or comment since no significant changes in practice have taken place since the date of the last report.

Cut-back asphalts are being used to a limited extent in penetration macadam and are particularly useful for work in cold weather. Materials of the RC–4 grade (see table 1) or even more viscous grades are used for this purpose. When cut-back asphalts are used it is desirable to use a coarse crushed aggregate of somewhat smaller size than is employed customarily when hot materials are used.

The use of emulsified asphalt for bituminous macadam construction has increased materially, and when it is used some changes in the standard construction procedure are required. In hot penetration construction the voids between the pieces of coarse stone are left open in order that the bituminous material may penetrate before it cools. When emulsified asphalt is used it is necessary to choke the voids with clean stone chips prior to the first application of bituminous material. This necessary modification of hot penetration practice was first used in Great Britain and Continental Europe. The subsequent application of keystone is usually followed by two light applications of emulsion, each covered with stone chips and rolled, and these in turn are followed by a light seal coat of emulsion that is covered with sand. In general, a somewhat smaller quantity of bituminous material is used for a given thickness of macadam constructed with emulsified asphalt than is usually considered necessary when asphalt cement or tar is used.

Low-cost surfaces.—The term "low-cost" as applied to roads and road surfaces has come into general use in the United States in recent years and its meaning is frequently misunderstood. Such roads and road surfaces are "low-cost" or not, depending entirely on one's conception of what constitutes a low cost. Actually they cost an appreciable sum per mile and an enormous sum in the aggregate. They are not inexpensive but merely less expensive than the higher types of pavement such as sheet asphalt, asphaltic concrete, or portland-cement concrete. But these low-cost surfaces, if constructed on adequate bases, are capable of carrying a large volume of motor-vehicle traffic, including an appreciable percentage of heavy trucks. Their use has made possible the construction of much greater mileages of surfaced roads, on the primary highway systems in sparsely populated areas, and on the secondary highway systems in more thickly populated areas, than otherwise would have been possible.

Low-cost bituminous surfaces, as used in the United States, may be divided into three general classes as follows:

1. Surface treatments.
2. Open-graded or coarse-aggregate surfaces.
3. Dense-graded surfaces.

The open-graded and dense-graded surfacing mixtures may be either mixed in place on the road, premixed in a traveling mixing plant, or premixed in a stationary mixing plant.

It is in these types of bituminous road surfaces that the greatest progress has been made since the Congress of 1930. In these types of construction the greatest amounts of liquid bituminous materials are used and the increased consumption of these products in recent years is an index of the extent to which the low-cost surfaces are being utilized. However, it should be noted that some of the road surfaces of this class are not resistant to the mutilative effect of steel-tired wheels and the hoofs of animals. The United States is fortunate in having very little highway traffic of this character.

Surface treatments or carpet coats are used on water-bound macadam, sand-clay, sand-clay-gravel, or other stable bases and on old bituminous surfaces. Widely varying materials are used and there is a considerable variation in construction procedures. However, disregarding minor refinements, the general practice is to prime the base with bituminous material and, after this has penetrated and dried, a heavier bituminous material is applied. This is covered immediately with clean coarse aggregate, generally having a maximum size of 1 inch or less, and this is rolled. Another coat of bituminous material is then applied and this is covered with stone chips and rolled. It is considered good practice to seal the resulting surface with another light application of bituminous material which is covered with chips and rolled. The seal may be applied at the time of construction or application may be delayed for several months. The net result is a carpet or mat having a thickness of about three-fourths inch and composed of bituminous binder and open-graded aggregate. The one thing common to all surface treatments is that the bituminous material is applied by means of a pressure distributor.

For prime coat, the lighter grades of tar or medium-curing cut-back asphalt are used, the grade selected being dependent on the density of the surface to be primed. The prime coat is sometimes omitted, particularly when surface treating old bituminous pavements or when using emulsified asphalt.

The bituminous materials applied after the prime coat are usually of heavier grades and may be hot asphalt cement, rapid-curing cut-back asphalt, tar or quick-breaking emulsified asphalt. The grade of material selected is dependent to a certain extent on weather conditions, the lighter grades being used in the spring and fall and those of higher viscosity in the summer. In connection with the growing tendency to use the heaviest grades of material that are suitable for the type of construction, attention should be called to the fact that the typical uses of the various grades of liquid asphaltic road materials, shown in table 1, are not entirely in accord with current practice. Great quantities of rapid-curing cut-back asphalt are used in surface treatment work and the RC–1 grade has been used extensively but there has also been a considerable use of the more viscous grades, even including RC–4.

The open-graded or coarse-aggregate surfaces are, as the name implies, composed of bituminous material and clean coarse aggregate free from dust. This type of surface is known by a variety of names, the most common of which is "retread treatment." It is used extensively, particularly for surfacing old gravel or macadam roads and for resurfacing old pavements that are worn or that have excessively high crowns which require flattening. The open-graded surfaces are used to the greatest extent in the eastern part of the United States.

As in the case of surface treatments and other types of bituminous construction, the details of construction vary somewhat in different States. In general, the procedure for road-mix or mixed-in-place construction is about as follows. After the base or old pavement surface has been prepared by removing loose dust, it may or may not, depending on its character, be given a prime treatment with light tar or cut-back asphalt. The coarse aggregate is then spread over the surface to the required depth and bituminous material is applied with a pressure distributor. The materials are then mixed with disk harrows, blade graders, or other simple mixing equipment, after which the mixture is spread and rolled. This is followed by an application of bituminous material and chips to choke the voids in the base mix and this, after being rolled, is given a seal coat of bituminous material, covered with chips or sand, and rolled.

It is frequently desirable to premix the coarse aggregate and bituminous material, either in a portable mixer at the site of the work or in a stationary mixing plant convenient to the supply of aggregate, rather than to mix it in place on the road. In such cases the details of construction, after the base mixture has been spread, are the same as for mixed-in-place work.

The thickness of open-graded surface courses varies from about 1¼ to 2½ inches and the size of the coarse aggregate used is dependent on this thickness. Typical requirements for size of aggregates are those of the American Association of State Highway Officials, shown in table 4. The size designations (34 B, 3 A Modified, etc.) given at the tops of the columns in table 4 are standard size designations of the Simplified Practice Recommendations for sizes of coarse aggregates for use in highway construction. In table 4 the same aggregate is specified for the key or choke stone and for the seal or cover stone. A size intermediate between that of the coarse aggregate and the cover stone is sometimes specified for the choke stone.

Rapid-curing cut-back asphalt, tar, and emulsified asphalt are all used in open-graded road-mixed or plant-mixed surface courses. In plant mixes it is advantageous to use heavier, less volatile grades of asphalt cutbacks and tars than are used in road mixes since these do not require as much aeration before the mixture is ready for compaction. Cut-back asphalt

Table 4.—*American Association of State Highway Officials proposed standard specification for crushed stone, crushed slag and crushed gravel for open-graded bituminous road-mix surface course. Size requirements for aggregates* [1]

Sieve size (square openings)	Coarse aggregate			Choke and seal
	34 B	3A modified	45 B	5B modified
	Percent	*Percent*	*Percent*	*Percent*
2-inch	100			
1½-inch	90-100	100		
1-inch		90-100	100	
¾-inch	30-65	40-75	90-100	
½-inch		5-20		100
⅜-inch	0-10		20-55	90-100
No. 4	0-5	0-5	0-10	10-30
No. 8			0-5	0-8

[1] Figures given are percentages passing the various sieves, by weight.

of grade RC-3 (table 1) is commonly used for road mixes in warm weather, RC-2 for road mixes in cold weather, and RC-4 for plant mixes. The tars used range in grade from RT-6 to RT-9 (table 2). When emulsified asphalts are used, those of the medium-breaking or slow-breaking types are used for mixing with the coarse aggregate and quick-breaking emulsion for the subsequent applications.

The dense-graded surfaces of the low-cost type were first developed and used extensively in the States of the Pacific Coast and they are still used to the greatest extent in the western half of the United States. They were utilized first to eliminate dust and reduce the loss of road metal from untreated fine-crushed rock and gravel surfaces and in the early stages of their development the slow-curing liquid asphaltic materials were used exclusively. Since then the use of asphalt cutbacks of the medium- and rapid-curing types has been found to be advantageous and now all three types of liquid asphaltic materials are used. Emulsified asphalts of the slow-breaking type are also being used but not in such great quantities as the other materials. As this type of construction has become standardized more attention has been given to the grading of the mineral aggregate and to the character of the fine mineral material which, if of unsatisfactory quality, is likely to cause failure.

The dense-graded surfacing mixtures of the low-cost type are mixed in place on the road, or mixed in traveling mixers or in stationary mixing plants. When mixed in place on the road the operations are much the same as in the case of the open-graded mixtures except that a great deal more manipulation is required. If rain happens to fall when mixing is in progress, the mixing operations may be extended greatly due to the necessity of reducing the moisture content to a low percentage. When the traveling mixing plant is used the graded aggregate, which is placed along the road in a windrow of proper size, is picked up and mixed by the machine and deposited behind it ready for spreading and compaction. As compared with road mixing, the traveling mixer is advantageous in permitting a better control of the proportioning of materials, in securing a more uniform mixture, and in eliminating much of the delay and expense caused by rain. The stationary mixing plant has all the advantages of the traveling mixer and, in addition, permits a much more accurate control of the proportioning of materials. Both types of mixers have the common advantage of permitting the use of asphaltic materials of greater viscosity.

Upon completion of the mixing operation the dense-graded mix is spread and compacted to a thickness of from 1½ to 3 inches. Compaction was formerly effected entirely by traffic accompanied by blading of the surface, but there is a growing tendency to supplement this by compaction with rollers. It is also the preferred practice to seal the compacted surface with a light application of bituminous material which is covered with chips or sand.

Typical requirements for grading of the aggregates are those of the American Association of State Highway Officials, shown in table 5. These grading requirements are supplemented by other requirements designed to prevent failures of the type that have taken place in the past. Failures in dense-graded surfaces of the low-cost type have been attributed to two principal causes: Lack of adhesion, in the presence of water, between the aggregate and the bituminous materials which are of relatively low viscosity; and the tendency of the fine-grained filler material to swell and become plastic when wet. In the proposed specifications of the American Association of State Highway Officials it is required that the aggregate, when mixed with the bituminous material that is to be used, shall be resistant to the action of water; that the portion of the aggregate passing the No. 40 sieve shall not have a plasticity index greater than 6; and that the bituminous mixture shall not swell unduly when subjected to the standard test for swelling.

Table 5.—*American Association of State Highway Officials proposed standard specification for crushed stone, crushed slag and crushed gravel for dense-graded bituminous road- and plant-mix surface course. Size requirements for aggregates* [1]

Sieve size (square openings)	Mix	Seal	Sieve size (square openings)	Mix	Seal
	Percent	*Percent*		*Percent*	*Percent*
1-inch	100		No. 4	45-65	
¾-inch	85-100		No. 10	30-50	0-5
½-inch		100	No. 200 [2]	5-10	0-2
⅜-inch		90-100			

[1] Figures given are percentages passing the various sieves, by weight.
[2] A smaller percentage passing the No. 200 sieve may be specified by the engineer.

There is no standard practice with respect to the liquid asphaltic materials used in dense-graded, low-cost surfaces. Slow-curing materials and medium- and rapid-curing cut-back asphalts are all used extensively, although at present there is probably a considerably greater use of the slow-curing products and the medium-curing cutbacks than of the rapid-curing materials. The consumption of cut-back products for this type of construction is increasing rapidly since they have a decided advantage over the slow-curing materials in that they result in mixtures that are tougher and more resistant to the action of water and the abrasion of traffic. When asphalt cutbacks are used it is necessary to select the type with respect to the available mineral aggregate since the rapid-curing materials cannot be mixed successfully with aggregates containing a high percentage of material passing the No. 200 sieve. Five percent passing the No. 200 sieve is usually considered the maximum amount of fine material if rapid-curing cutbacks are used and if greater amounts of the fine material are present it is necessary to employ the medium-curing products.

In addition to the extensive use of dense-graded road surfaces containing liquid asphaltic materials, there is a more restricted use in the eastern States of the same general type of construction in which the graded aggregate is mixed with tar.

Stabilized bases.—A major development, since the Congress of 1930, has been in the design and construction, with soil materials (gravel, sand, silt, and clay), of the so-called stabilized bases that are suitable for bituminous surfacing. The use of soils in highway construction is the subject of another report to the Congress and therefore a detailed discussion of this topic is not appropriate here. However, it is necessary to mention stabilized bases since bituminous materials are being used in their construction in an ever increasing amount.

Given a supply of granular materials of suitable grading and fine soil binder of satisfactory quality, it is possible to proportion these materials for base-course construction so that satisfactory performance in service will be assured. But when the granular materials available are insufficient in quantity or of inadequate grading, or the soil binder is of unsatisfactory quality, it is necessary to overcome these defects by some means and bituminous materials have proven effective.

Emulsified asphalt of the slow-breaking type has been used more extensively in base stabilization than the other bituminous materials but tar and slow-curing, medium-curing, and rapid-curing liquid asphalts have also been used with success. However, bituminous base stabilization must still be considered as largely in the experimental stage.

In this type of construction the usual practice is to pulverize the soil material that is to be stabilized, add bituminous material in one or more applications, and mix it on the road. On account of the large amount of fine-grained soil and the relatively small amount of bituminous material used, it has been found that a uniform mixture can be obtained only by adding considerable quantities of water to the mix. This is necessary even with emulsified asphalt. The mixing is done with harrows and blades and the resultant mixture has the appearance and consistency of soft mud. It is spread, permitted to dry to a moisture content that will permit compaction, and is then compacted with sheepsfoot rollers or multiple-wheel rollers with pneumatic tires. Rollers of this type are preferred since they produce the kneading action, so essential to thorough compaction, that cannot be secured with ordinary road rollers.

In addition to the road-mix method that has been described, mixing is also done with traveling mixers of the same type as those used in the preparation of surfacing mixtures.

In the road-mix method, the bituminous materials are always applied with pressure distributors except when a machine known as a sub-oiler, designed especially for the purpose, is used. This machine introduces the liquid bituminous material into the pulverized soil by means of pipe lines attached to scarifier teeth.

The stabilized soil layer thus produced is generally too friable to withstand the action of traffic and it is customary to protect it with a bituminous surface, usually one of the low-cost types.

CONSTRUCTION EQUIPMENT

A major development in connection with bituminous road construction in the United States has been the improvement of old types of construction equipment and the development of many new types. The introduction of new machines designed for special purposes has resulted in better and more economical construction and has had an important influence on present practices.

General improvements, common to a number of types of equipment, have been the substitution of pneumatic tires for steel or solid-rubber tires, the adoption of Diesel-type power units where practicable, a change from mechanically operated controls to those designed for compressed-air or hydraulic operation, and a general increase in capacity of all equipment.

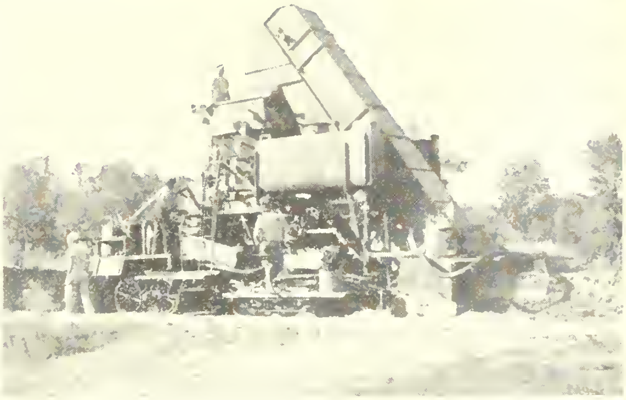
BITUMINOUS DISTRIBUTOR WITH FULL-WIDTH SPRAY BAR AND TACHOMETER ACTUATED BY AN AUXILIARY WHEEL.

Asphalt plants.—Stationary plants for the manufacture of hot asphaltic paving mixtures have been greatly improved both from the standpoint of design and that of control of operations. The operation of the modern plant is largely automatic, a recent improvement being temperature control of the aggregates in the drier by means of a pyrometer that regulates the burners. This insures delivery of the aggregate to the hot elevators at a more constant temperature with a resultant improvement in the uniformity of the mixing operation. The automatic feeding of aggregates to the drier is becoming common practice. In the past the asphalt heaters used at asphalt plants have usually been of the oil-burning type but an electric heater has been developed that can be used to advantage where the cost of electric power is low. The new electric heater is automatic and will control the temperature of a tank of asphalt within narrow limits, thus reducing the danger of damaging the material by overheating.

Automatic plant control has brought about a great improvement in the quality of the product and has also contributed materially to increased production which has resulted in a lowering of costs.

Asphalt heaters and distributors.—Bituminous road materials are usually shipped in railroad tank cars having a capacity ranging from 6,500 to 10,000 gallons and these cars are equipped with coils for the purpose of heating the material when necessary. The low-pressure steam heaters formerly used are gradually being discarded in favor of high-pressure steam heaters that are more efficient and more easily portable. With this equipment it is possible to raise the temperature of the material in a 10,000-gallon tank car as much as 50° F. in one hour. The shipment from the refineries of heated materials in insulated tank cars and the use of the improved heaters has greatly reduced the time required to transfer material from the tank car to the distributor.

Pressure distributors for the application of bituminous materials are employed in all types of bituminous construction that do not involve plant mixing. The sizes most commonly used have capacities ranging from 600 to 1,500 gallons. The general design of distributors has not changed greatly but refinements in design have increased the ease and efficiency with which materials are handled. More uniform distribution is made possible by increased capacity of pumps and by better heating and circulating systems. Accuracy in the rate of distribution is aided by tachometers,



Traveling Mixing Plant that Picks up the Mineral Aggregate from a Windrow, Mixes it with Bituminous Material, and Deposits the Finished Mixture Behind the Machine.

actuated by an auxiliary wheel, or by other forms of governors that synchronize the speed of the pump and the speed of the truck. Another improvement is a new spray-bar that eliminates dripping as soon as the flow is stopped. This is accomplished by applying suction from the pump or by cut-off valves for each nozzle.

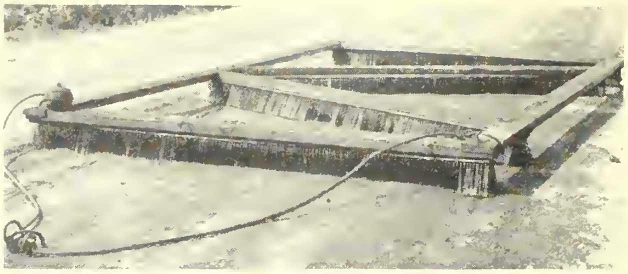
Portable mixing and spreading equipment.—For mixed-in-place construction the greatest use is made of such simple equipment as spring-tooth harrows, disk harrows, and blade machines. An improvement in the disk harrow has been made by mounting the disks in a frame carried on rubber-tired wheels. This permits the disks to be lifted clear of the ground and is advantageous in turning the equipment. The grader machine with a single blade and the multiple-blade machine are both used extensively in road-mixing operations. The multiple-blade machines have four or more blades set in pairs for mixing and for striking off. An improvement in this equipment is obtained by setting the blade assembly in a chassis mounted on rubber-tired wheels, with provision for quickly raising and lowering the blades.

Several types of traveling mixing machines are now available and their use is increasing rapidly. In one type, one or more pug-mill shafts are installed at the front of the machine and, as it moves forward, mixing of the aggregate and bituminous material, previously distributed on the road, is effected in one passage. In another type the aggregate, previously placed in a windrow on the road, is picked up and elevated to a continuous mixer. There the bituminous material is added and mixed, and the finished mixture is then deposited on the road in the rear of the machine.

For the types of construction in which the aggregate is placed in windrows along the road, it is desirable that the windrow be of uniform cross-section and that it contain the required amount of material. A useful device for this purpose is dragged by a tractor over the roughly formed windrow, bringing it to uniform section by means of adjustable tail gates and moving excess material forward.

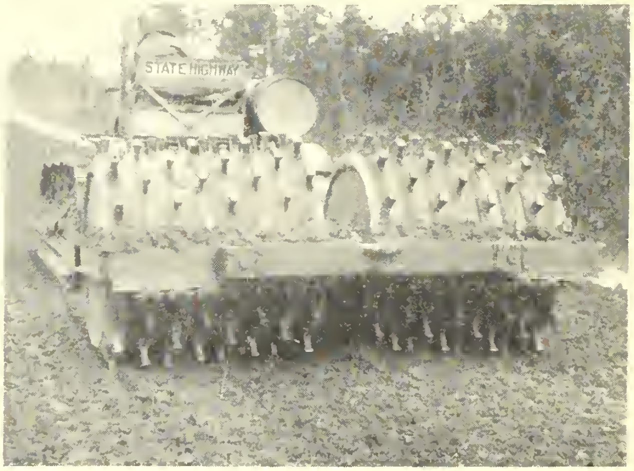
Brooms and broom drags.—For cleaning road surfaces prior to bituminous construction, better and speedier rotary brooms have been developed and these, combined with motor-driven blowers, make it possible to clean a surface more effectively and economically than can be done with hand brooms.

For spreading small-size aggregate in macadam construction or in surface-treatment work, or for mixing and spreading small quantities of bituminous material and small-size aggregate, the broom drag has been found very useful in effecting uniform distribution and smoothness of surface. These drags are fairly large and contain several rows of stiff-fiber or wire brooms.



A Broom Drag.

Aggregate spreaders.—Mechanical spreaders, that insure uniformity of distribution of aggregate, are practically always used in the construction of penetration macadam and surface treatments and they are also used in some types of road-mix construction. These are of various types and forms. In one type the aggregate is spread directly from the rear of a dump truck by spreader devices built into the tail gate or attached to it. A second type includes hoppers of various forms, either pulled by a truck or self-propelled, that may be adjusted to spread a layer of aggregate in any amount desired. A third, and rather novel type, consists of two small wheels on an axle that carries, and is geared to, a revolving circular steel plate. This device is attached to the rear of a dump truck loaded with aggregate. As the truck moves forward the aggregate is permitted to fall through the tail gate of the truck onto the revolving plate which sprays it over the road surface. This type is designed for relatively light applications of aggregate in surface treatment.



A Sheepsfoot or Tamping Roller Which Is Very Effective in Compacting Bituminous-Stabilized Bases that Contain a High Percentage of Fine-Grained Material.

Rollers.—For the compaction of bituminous-stabilized bases and bituminous mats the ordinary construction rollers having wide, smooth steel wheels are not generally satisfactory and three special types of rollers, none of which is self-propelled, are used for this purpose. One of these is the sheepsfoot or tamping roller which is not new but which has not been used extensively until recent years. This roller consists of a hollow metal drum, which may be filled with water, and which is studded on its outer surface with metal projections.

These studs are about 6 inches long, of small diameter and end in flat heads somewhat larger than the shanks. These rollers are very effective in obtaining uniform compaction throughout the entire depth of layers of fine-grained soil materials.

A second type is a multiple-disk roller in which 10 or more heavy metal disks, about 3 feet in diameter and 2 inches thick, are mounted a few inches apart on a single axle. The third type is the multiple-wheel pneumatic-tired roller. This is a trailer having either one or two axles, on each of which are mounted, at close spacing, as many as 5 truck wheels equipped with large pneumatic tires. The trailer body may be loaded with any desired weight. This roller is very effective in producing the kneading action that is necessary in compacting bituminous-stabilized bases and dense-graded bituminous surfaces.

A valuable improvement has been made in the conventional tandem and three-wheel rollers that are used so extensively in bituminous pavement construction. This is the addition of an auxiliary roller on a third axle. In the case of the tandem machine the auxiliary roller is of about the same size as the forward roller and is placed in front of it. In the three-wheel machine the auxiliary roller is of considerably smaller diameter than the main rollers and may be placed either between the front and rear axle or behind the rear axle. The auxiliary roller is adjustable as to height and is very effective in eliminating surface irregularities and in producing smoother pavements.

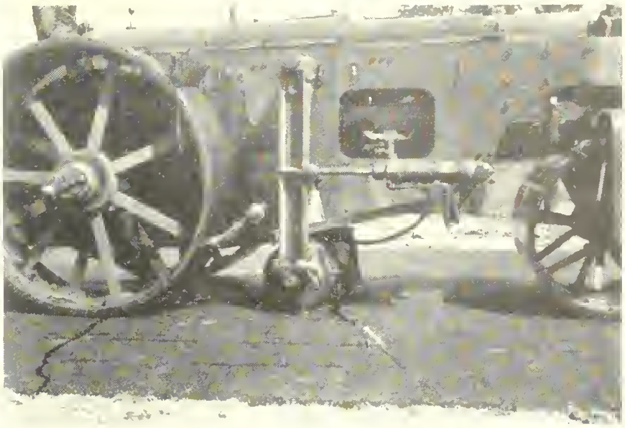
Finishing equipment.—One of the important developments in the construction of plant-mixed bituminous surfaces has been the introduction of mechanical equipment for spreading and leveling the mixture preparatory to rolling. These finishers are of three principal types: The first type is supported on side forms of wood or steel; the second provides its own forms, being supported on long skids or runners that carry the machine over slight depressions and minimize the effect of these irregularities in the base; and the third is carried on rollers of moderate diameter.

The bituminous mixture may be deposited on the road from a hopper that is a part of the finishing machine or it may be dumped from trucks and shoveled into place by hand. One type of hopper contains a horizontal shaft equipped with blades that agitate the mix, thus breaking up lumps and preventing segregation. The mix then flows through the bottom of the hopper and falls in front of the screeds that spread it to a uniform thickness.

Finishing machines are almost always power driven and many of them have an attachment for heating the screed so that the mixture will not cool too much during the spreading process. Some machines have two or three screeds and in the latter case the middle screed is equipped with teeth that rake and loosen the mixture prior to the passage of the last screed. The selection of the type of finisher most suitable for the work depends somewhat on the character of the bituminous mixture but it is becoming common practice to specify the use of some type of mechanical finishing equipment, particularly for the higher types of pavement such as sheet asphalt and asphaltic concrete.

LABORATORY RESEARCH AND FIELD INVESTIGATIONS

The asphaltic road materials used in the United States are produced largely from petroleum that is obtained from a great number of sources and refined by a variety of processes. Petroleum from the different sources varies greatly in character and crude oil from the same source may yield asphaltic materials of widely different character, depending on the method of refining. The cracking processes, designed to increase the yield of gasoline, are used extensively and these produce asphaltic residues that are not always of satisfactory quality.



Three-Wheel Road Roller With Auxiliary Adjustable Roller.

Early in the period that has seen such a great increase in the consumption of liquid asphaltic materials, failures of the lower-cost and lower-type surfaces in which they are used aroused interest in their quality. This interest has been extended, with good reason, to asphalt cements that, in the previous period, had not been the object of serious suspicion. With respect to road tars a new problem has been introduced by the greatly increased production of heavy water-gas tars from heavy carbureting oils of what are known as the "residuum" type. These have largely replaced the light water-gas tars formerly produced.

Experience has shown that one of the most important properties of a bituminous road material is its durability or resistance to weathering, and it is recognized that there are no existing specification requirements that insure this property. As a result a number of agencies have concerned themselves with research work designed to develop some test for predetermining the durability of bituminous materials. These studies have included the exposure of test samples to atmospheric weathering in the open and to the action of heat, light, water, and freezing temperatures in the laboratory and the attempt to correlate the results with the behavior of roads in service. Several reports of work that has been done are listed in the bibliography (*2*).

The dissatisfaction with asphaltic materials that have been injured in refining, either by overheating or by being subjected to cracking processes has resulted in the development of the Oliensis spot test (*3*) and the test for xylene equivalent (*4*). The Oliensis spot test is not claimed to be a test of quality. It is merely an arbitrary measure of heterogeneity and serves to identify those materials that, through cracking or overheating, have been rendered sufficiently heterogeneous to react to the test. A number of State highway departments that have had trouble with cracked materials are using this test to exclude them. The xylene equivalent is a modification of the Oliensis test designed to give a quantitative measure of the degree of heterogeneity.

The cracking of sheet asphalt and asphaltic concrete pavements is a problem that is engaging the attention of a number of investigators. Studies have been and are being made of the characteristics of these paving mixtures at low temperatures, as disclosed by tests for toughness, shearing strength, compressive strength, tensile strength, and modulus of rupture and modulus of elasticity in flexure. Also the hardness and ductility of asphalts extracted from existing pavements are being correlated with the prevalence of cracking. Recent reports of investigations of this character are listed in the bibliography (*5*).

Since hardening of the asphalt in a paving mixture has been found to reduce the resistance to cracking, considerable attention is being given to studies of the changes that take place in the asphalt during the operations of mixing and placing (*6*). It has been found that both the time of mixing and the temperature of the materials have important effects on the hardening and decrease in ductility that occur and that different asphalts are affected to different degrees. Such investigations, which involve the extraction and recovery of asphalt from paving mixtures, have been aided greatly by the recovery methods of Abson and Bussow (*7*).

Numerous studies are being made of the resistance, in bituminous mixtures, of the stripping of the bituminous films from the mineral aggregate in the presence of water. In the field of base stabilization with bituminous materials the studies of Winterkorn (*8*) in surface chemistry are being continued.

Numerous field investigations, involving the construction and observation of experimental roads, are being conducted. Included in these are numerous investigations of the stabilization of base courses with bituminous materials. Also studies are being made of the use, in asphaltic concrete pavements, of softer asphalts than are generally considered suitable. A great deal of publicity has been given to the use, in the United States, of cotton fabric for the reinforcement of bituminous road surfaces. Several hundred miles of experimental roads have been built in which surface treatments, and similar types of light bituminous surfacing, have been reinforced with coarsely woven cotton fabric. However, these experimental roads have not been in service long enough to demonstrate the merits of the reinforcement.

SUMMARY

1. Since the Congress of 1930 the use of bituminous road materials in the United States has increased greatly. This increased consumption is accounted for entirely by the increased use of liquid products in bituminous road surfaces of the low-cost type and in the construction of bases for these surfaces.

2. The low-cost types of bituminous surfaces are: Surface treatments, open-graded surfaces, and dense-graded surfaces. Great progress has been made in the development and improvement of these types.

3. Liquid asphaltic products, tars, and emulsified asphalts are all used successfully in the low-cost types of surfaces.

4. Many of the improvements that have been made are the direct result of the development of numerous new types of construction equipment.

5. Important progress has been made in the design and construction, with soil type materials, of bituminous stabilized base courses suitable for low-cost bituminous surfaces. To a great extent the use of bituminous products in stabilizing soil materials for base course construction is still in the experimental stage. However, emulsified asphalt has been used with notable success, as well as tar, slow-curing liquid asphalt, and cutback asphalt.

6. The quality of bituminous road materials, particularly with respect to their durability, is not assured by existing specifications. Information on this question is being sought in a number of different investigations.

ACKNOWLEDGMENTS

This report is based in large part on a number of special reports concerning the present practice in the use of bituminous materials in road construction in the United States. Grateful acknowledgment is made to the following engineers who furnished these reports.

Shreve Clark, Engineer of Tests, Virginia Department of Highways.

Bernard E. Gray, Chief Highway Engineer, The Asphalt Institute.

Prevost Hubbard, Chemical Engineer, The Asphalt Institute.

George E. Martin, Consulting Engineer, General Tarvia Department, The Barrett Co.

C. L. McKesson, Director of Research and Engineering, American Bitumuls Co.

J. T. Pauls, Senior Highway Engineer, United States Bureau of Public Roads.

C. S. Pope, Construction Engineer, Division of Highways, California Department of Public Works.

Dr. Lloyd F. Rader, Associate Professor of Civil Engineering, Polytechnic Institute of Brooklyn.

Dr. E. O. Rhodes, Technical Director, Tar and Chemical Division, Koppers Co.

A. C. Tilley, State Engineer, Nebraska Department of Roads and Irrigation.

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(*Continued from page 57*)

applies particularly to the mixing and distributing units which in their present form cannot handle concrete of much lower slump than 1 inch. This phase of the problem will unquestionably receive attention in the near future.

Curing.—For the curing of concrete pavements, cotton mattresses, composed of cotton "bats" covered with cotton cloth, have recently been employed to a considerable extent.[7] This type of covering material, in addition to performing the usual function of a curing medium by protecting the surface against loss of moisture and supplying needed moisture to the surface, has the ability to insulate the concrete against extreme changes in temperature during the curing period. Field experiments are now being conducted in various parts of the United States for the purpose of determining the practicability of this type of mattress or mat in the actual construction process.

Additional equipment and construction methods.—There has been no marked recent change in the type of equipment used in handling and transporting materials from the proportioning plant to the mixer. Locomotive or caterpillar cranes are used in moving the materials from freight cars to overhead proportioning bins. Trucks hauling from one to five batches, including cement, transport the materials to the mixer. Industrial narrow-gage railroads are now employed only where trucking is impracticable, such as over sandy subgrades.

Dual-drum mixers have been developed, whereby the batches are mixed in one drum for about half of the mixing period and then transferred to the other for the remainder of the period. With this equipment it has been found possible to increase production about one-third over that of a single-drum mixer. Occasionally, where conditions are favorable, two mixers have been operated at the same location. In some States of the United States, the hauling of materials over the subgrade ahead of the mixer and the operation of the mixer on the subgrade are not permitted, except where such procedure is unavoidable.

Recently a power-driven longitudinal float has been developed, which is operated back of the finishing machine, taking the place of the hand-operated longitudinal float. It is claimed that a more nearly perfect crown and surface can be produced with it.

Internal vibrators of the spud type have come into use for consolidating concrete around joints and adjacent to the forms.

II. CEMENT-BOUND MACADAM

After having fallen into disuse, the construction of cement-bound macadam has again returned to use mainly as a means of employing large numbers of otherwise unemployed men.

In 1933 the Portland Cement Association undertook a laboratory and field investigation to determine the requirements for coarse aggregate and grout in this work, and to study construction methods. A suitable test for fluidity of grout, the "flow cone" test, was developed, and the necessary properties of grout for various sizes of coarse aggregate were determined. Both gravel and crushed stone coarse aggregate were used successfully and depths up to 10.6 inches were effectively penetrated with grout, without resorting to any special method to facilitate the flow. Compaction of gravel coarse aggregate before grouting was found impracticable, and compaction of crushed stone reduced the grout consumption. Satisfactory results were obtained with hand tamping.

The tests showed that compaction after grouting, when properly timed, increased the strength of the slab by reducing voids left by escaping water. Heavy rollers, however, forced some of the coarse aggregate into the subgrade, which had been softened by water from the grout, and caused irregularities in the surface which were difficult to remove. Compaction by hand tamping gave slightly lower strengths, but loss of material into the subgrade was eliminated. Compaction by vibration also gave good results, but the available equipment was not practical.

Employing the information thus made available, appreciable quantities of cement-bound macadam have since been laid, largely as a measure of work relief for the unemployed One development in the field has been the successful use of broken brick and concrete from old pavements as a part of the coarse aggregate.

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[7] Public Roads, Vol. 14, No. 5, July 1933; and Vol. 15, No. 9, November 1934.

III. CEMENT-HARDENED EARTH ROADS

With the objective of developing a material suitable for low-cost, all-weather, secondary road surfaces, as well as methods of construction, rather extensive laboratory and field experimentation with soil-cement mixtures has been carried on during the last few years. As a general rule, this work has been done by the Portland Cement Association in cooperation with various States and the United States Bureau of Public Roads.

While this type of construction is by no means out of the experimental stage, definite principles governing soil-cement mixtures have been established and experience has shown that laboratory work must precede the field work in order that any degree of success may be assured.

Each field job has simplified the application of laboratory results with attendant economies of construction. The surfaces constructed are capable of withstanding a considerable amount of traffic. The projects constructed to date have gone through one winter without structural disintegration.

(Continued from page 60)

pavement slab was installed in the driveway entrance to the plant of a paving-brick company in Illinois. Brick were placed in a "basket weave" manner with three brick constituting a unit. Reinforcement, consisting of ⅜-inch round deformed bars was placed at 8¾-inch centers both ways, 1 inch from the bottom, and all joints were filled with 1: 2 cement mortar. Two weeks after the slab was completed an excavation was made under one side creating a clear span of 5 feet. It is estimated that during the past 6 years 25,000,000 paving brick have been trucked over the pavement which shows no sign of distress over the excavation.

Encouraged by the success of this experiment, additional test installations are now being planned in Ohio and Illinois. Instead of using standard paving brick in basket weave pattern, large vitrified units 3½ by 8 by 8-inches have been manufactured. They will be laid by hand in checkerboard pattern, properly spaced, and reinforcing rods will be placed in two directions in the cement-grouted joints.

MUNICIPAL USE

Vitrified brick are widely used as a paving material by municipalities in the United States. Engineers generally regard it as a superior type of surface for streets carrying heavy and destructive traffic, and it is frequently used for all types of streets in districts close to brick plants.

Many kinds of bases are used in cities but as in rural road practice, concrete is most popular over the entire country. Mixes vary but cities seldom use a mix richer than 1 part cement, 2½ parts fine aggregate, and 5 parts course aggregate while the leaner proportioning known as the 1 : 3 : 6 mix is more widely used at present. A few cities use a 1 : 2 : 4 mix. Depths vary from 5 to as much as 10 inches, the popular average being 6 inches. There seems to be little, other than individual preference, to account for the wide variation in practice.

Beds or cushions used in municipal practice are also similar to those used on rural highways. However, in addition to the mastic beds, many cities are still successfully using plain sand.

In municipal work a 3-inch depth brick is generally used and a wire cut surface is exposed to traffic to avoid slipperiness. The rather complete motorization of all services in American cities with the resulting elimination of the horse has largely done away with the hillside brick although some are still used.

The type filler in most general use in cities is a so-called hard asphalt with low penetration and a high melting point. Use of the softer asphalts has resulted in a substantial loss of the filler from the joints in hot weather and slipperiness caused by the adherence of the filler material to the wearing surface of the brick. Use of the hard asphalts does not entirely cure this trouble and materials of the experimental types previously mentioned are being tried for city as well as rural use.

In recent years many old brick pavements in cities have been relaid, either on the old bases or on new foundations, which results in appearance and service value approaching those obtainable with entirely new construction. As the removing and cleaning of the old brick before relaying is entirely a labor item, the cost varies with the wages paid. The cost of cleaning is also dependent upon the type of filler used in the original construction, varying between the extremes of plain sand and rich cement grout. In most of the old pavements the brick were laid on edge. When relaid they are frequently laid flat, according to present-day practice, and there is a gain in surface area. This fact affects the percentage of salvage measured by the area covered by the relaid brick. In some cases this has actually resulted in a salvage greater than 100 percent and a surplus of brick which were used for additional paving. However, the average proportion of salvaged brick is about 80 percent. Owing to the increase in manufacturing costs since the brick pavements were originally constructed years ago, the relay value of the brick is in many cases materially greater than the original cost.

STATE MOTOR-FUEL TAX RECEIPTS, 1937

(Compiled for calendar year from reports of State authorities)

State	Tax rate per gallon		Date of rate change	Receipts from taxation of motor fuel			Other receipts in connection with motor-fuel tax					Net total receipts
	On Jan. 1	On Dec. 31		Gross receipts	Refunds paid	Net receipts	Distributors' and dealers' licenses [1]	Inspection fees [2]	Fines and penalties	Miscellaneous receipts [3]	Total	
	Cents	*Cents*		*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*	*1,000 Dollars*
Alabama	6	6		13,295		13,295		55			55	13,350
Arizona	5	5		5,059	734	4,325	(*)		8		8	4,333
Arkansas	6½	6½		9,877		9,877		85			85	9,962
California	3	3		51,486	4,872	46,614	10				10	46,624
Colorado	4	4		8,659	1,228	7,431						7,431
Connecticut	3	3		9,640	195	9,445	51				51	9,496
Delaware	4	4		2,173	142	2,031	3				3	2,034
Florida	7	7		22,023		22,023	49	394			443	22,466
Georgia	6	6		19,550		19,550						19,550
Idaho	5	5		4,532	497	4,035				2	2	4,037
Illinois	3	3		38,907	3,071	35,836		428	2		430	36,266
Indiana	4	4		24,699	1,712	22,987	(*)	509		1	510	23,497
Iowa	3	3		14,836	1,813	13,023	1	23			24	13,047
Kansas	3	3		10,083		10,083	6	104		43	153	10,236
Kentucky	5	5		12,671		12,671						12,671
Louisiana	7	7		15,923		15,923		75	2	(4)	77	16,000
Maine	4	4		5,755	205	5,550						5,550
Maryland	4	4		10,587	730	9,857						9,857
Massachusetts	3	3		20,712	876	19,836						19,836
Michigan	3	3		30,774	1,349	29,425	4			1	5	29,430
Minnesota	3	4	Apr 23	17,594	2,301	15,293	1	199			200	15,493
Mississippi	6	6		10,801	579	10,222				(5)		10,222
Missouri	2	2		11,675	593	11,082		121	11		135	11,217
Montana	5	5		5,421	843	4,581						4,581
Nebraska	5	5	(6)	11,157	232	10,925		105			105	11,030
Nevada	4	4		1,248	71	1,177	(*)					1,177
New Hampshire	4	4		3,385	99	3,286						3,286
New Jersey	3	3		23,581	2,019	21,565	26			2	28	21,593
New Mexico	5	5		4,447	444	4,003	19				19	4,022
New York	3	4	May 10	63,569	1,728	61,841	63			11	74	61,915
North Carolina	6	6		22,950	521	22,429		958		6	964	23,393
North Dakota	3	3		3,453	580	2,873	1	61			62	2,935
Ohio	4	4		[7] 48,639	2,101	46,538	(*)					46,538
Oklahoma	4	4		15,050	1,282	13,768			4		4	13,772
Oregon	5	5		11,161	1,362	9,799				2	2	9,801
Pennsylvania	4	4		55,715	4	55,711			4	5	9	55,720
Rhode Island	2	3	Apr. 22	3,322	232	3,090	4				4	3,094
South Carolina	6	6		11,160	259	10,901		234			234	11,135
South Dakota	4	4		4,380	309	4,071		84			84	4,155
Tennessee	7	7		17,914		17,914		1,024			1,024	18,938
Texas	4	4		47,895	6,224	41,671				7	7	41,678
Utah	4	4		3,421		3,421	2		1		3	3,424
Vermont	4	4		2,323		2,323						2,323
Virginia	5	5		17,112	998	16,114	(*)		7	1	8	16,122
Washington	5	5		16,558	1,290	15,268	2			12	14	15,282
West Virginia	4	5	Apr. 1	8,695	207	8,488	8				8	8,496
Wisconsin	4	4		21,032	1,495	19,537		214			214	19,751
Wyoming	4	4		2,498		2,498	2				2	2,500
District of Columbia	2	2		2,737	13	2,724	8				8	2,732
Total	Weighted average rate 3.91 cents			800,140	43,210	756,930	260	4,676	39	93	5,068	761,998

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[1] Stars (*) indicate amounts less than $500.

[2] Fees for inspection of motor-vehicle fuel. Wherever possible fees for inspection of kerosene and other non-motor-vehicle fuels have been eliminated.

[3] Includes fees for motor-fuel carrier permits, refund or exemption permits, interest on deposits, and miscellaneous unclassified receipts.

[4] Receipts from tax on lubricating oil, $570,000, not included in this table.

[5] Special taxes of 3 cents per gallon in Hancock County and 2 cents per gallon in Harrison County are imposed for sea-wall protection. The receipts from these taxes, $151,000, in 1937, are not included in this table.

[6] Tax was 5 cents Jan. 1 to Feb. 28; 4 cents Mar. 1 to 24; 5 cents Mar. 25 to Dec. 31.

[7] Ohio imposes a 3-cent tax on motor-vehicle fuel and a 1-cent tax on all liquid fuels. The receipts from the 1-cent tax applicable to non-motor-vehicle fuels (kerosene, fuel oil, etc.) were $662,000. These receipts have been eliminated from the total given, which represents a 4-cent tax on motor-vehicle fuel.

MOTOR-FUEL CONSUMPTION, 1937

(Compiled for calendar year from reports of State authorities)

State	Tax rate per gallon		Date of rate change	Motor-fuel consumption as reported by States								Analysis of motor-fuel usage [5]						
								Net amount taxed [3]								Highway use		
										At reduced rates								
	On Jan. 1	On Dec. 31		Gross amount reported [1]	Amount exempted from payment of tax [2]	Gross amount Assessed for taxation	Amount Subject to refund of entire tax	Total	At full rate [4]	Rate per gallon	Amount	Net total Consumption in State	Allowance for evaporation and other losses [6]	Federal and other public use [7]	Nonhighway use [8]	1937	1936 [9]	Percentage change
	Cents	Cents		1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	Cents	1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	1,000 gallons	
Alabama	6	6	-----	222,121	-----	222,121	-----	222,121	222,121	-----	-----	228,954	*3,382	*3,451	*11,448	210,673	188,931	11.5
Arizona	5	5	-----	104,312	5,786	98,526	12,130	86,396	86,396	-----	-----	104,312	995	4,791	12,130	86,396	78,304	10.3
Arkansas	6½	6½	-----	164,146	6,059	158,087	-----	158,087	142,383	(10)	15,704	164,146	1,535	4,524	*8,207	149,880	140,566	6.6
California	3	3	-----	1,762,039	38,379	1,723,660	162,402	1,561,258	1,561,258	-----	-----	1,762,039	8,414	29,965	162,402	1,561,258	1,464,072	6.6
Colorado	4	4	-----	225,498	10,333	215,165	30,364	184,801	184,801	-----	-----	225,498	4,615	9,555	26,527	184,801	172,046	7.4
Connecticut	3	3	-----	326,907	6,883	320,024	6,496	313,528	313,528	-----	-----	326,907	3,228	1,341	8,810	313,528	287,242	9.2
Delaware	4	4	-----	54,849	1,041	53,808	2,930	50,878	50,878	-----	-----	55,123	274	1,041	2,930	50,878	47,677	6.7
Florida	7	7	-----	329,646	13,001	316,645	-----	316,645	316,645	-----	-----	329,646	2,777	7,747	*16,482	302,640	273,070	10.8
Georgia	6	6	-----	331,760	6,491	325,269	-----	325,269	325,269	-----	-----	331,760	2,704	3,787	*16,588	308,681	283,475	8.9
Idaho	5	5	-----	95,087	4,350	90,737	9,911	80,826	80,571	2½	[11] 255	95,087	-----	3,859	10,658	80,570	74,700	7.9
Illinois	3	3	-----	1,293,364	-----	1,293,364	101,785	1,191,579	1,191,579	-----	-----	1,346,128	*20,098	*6,271	101,785	1,217,974	1,143,272	6.5
Indiana	4	4	-----	634,136	21,150	612,986	42,800	570,186	570,186	-----	-----	634,136	*18,958	2,192	42,800	570,186	520,103	9.6
Iowa	3	3	-----	495,896	-----	495,896	66,306	429,590	429,590	-----	-----	520,940	*15,337	*9,845	66,168	429,590	410,306	4.7
Kansas	3	3	-----	460,768	127,270	333,498	-----	333,498	333,498	-----	-----	470,171	*9,403	*2,720	124,550	333,498	316,438	5.4
Kentucky	5	5	-----	251,850	-----	251,850	-----	251,850	251,850	-----	-----	258,195	*3,188	*3,157	*12,910	238,940	216,502	10.4
Louisiana	7	7	-----	238,695	5,554	233,141	2	233,139	226,297	(12)	6,842	238,695	[12] 160	5,396	*11,935	221,204	201,392	9.8
Maine	4	4	-----	145,443	960	144,483	-----	144,483	137,649	1	[13] 6,834	146,903	*1,460	960	6,834	137,649	127,306	8.1
Maryland	4	4	-----	270,188	4,329	265,859	17,731	248,128	246,036	3	[14] 2,092	270,188	-----	4,329	17,731	248,128	226,128	9.7
Massachusetts	3	3	-----	696,473	3,723	692,750	28,189	664,561	664,561	-----	-----	696,473	-----	3,861	28,051	664,561	625,629	6.2
Michigan	3	3	-----	1,110,604	91,040	1,019,564	44,777	974,787	974,422	1½	[15] 365	1,110,604	31,533	11,764	94,368	972,939	868,947	12.0
Minnesota	3	4	Apr. 23	514,112	19,568	494,544	61,401	433,143	433,143	-----	-----	514,112	15,291	3,346	62,332	433,143	404,065	7.2
Mississippi	6	6	-----	186,839	8,111	178,728	-----	178,728	167,175	-----	[16] 11,553	186,839	3,626	4,485	11,553	167,175	151,879	10.1
Missouri	2	2	-----	590,669	-----	590,669	24,055	566,614	566,614	-----	-----	612,936	*18,268	*3,999	24,055	566,614	548,168	3.4
Montana	5	5	-----	113,800	6,162	107,638	16,655	90,983	90,983	-----	-----	113,800	2,153	4,009	16,655	90,983	90,459	.6
Nebraska	5	5	(17)	231,360	8,891	222,469	19	222,450	222,450	-----	-----	231,360	7,086	1,809	*23,136	199,329	200,064	−.4
Nevada	4	4	-----	33,781	1,811	31,970	1,767	30,203	30,203	-----	-----	33,781	-----	1,495	2,083	30,203	28,052	7.7
New Hampshire	4	4	-----	85,295	-----	85,295	2,488	82,807	82,807	-----	-----	87,325	*862	*1,168	2,488	82,807	78,608	5.3
New Jersey	3	3	-----	797,055	3,136	793,919	67,273	726,646	726,646	-----	-----	797,055	-----	12,671	22,743	761,641	696,746	9.3
New Mexico	5	5	-----	95,251	6,568	88,683	8,883	79,800	79,800	-----	-----	95,251	1,583	4,985	8,883	79,800	69,445	14.9
New York	3	4	May 10	1,815,563	61,029	1,754,534	51,913	1,702,621	1,702,621	-----	-----	1,815,563	-----	56,447	56,495	1,702,621	1,613,016	5.6
North Carolina	6	6	-----	391,816	8,070	383,746	-----	383,746	373,317	1	[16] 10,429	391,816	3,799	*3,650	11,050	373,317	335,097	11.4
North Dakota	3	3	-----	117,943	1,532	116,411	27,764	88,647	88,647	-----	-----	117,943	-----	1,532	27,764	88,647	79,570	11.4
Ohio [18]	4	4	-----	1,308,748	74,266	1,234,482	12,442	1,222,040	1,167,989	1	[19] 54,051	1,308,748	38,180	5,617	96,962	1,167,989	1,076,854	8.5
Oklahoma	4	4	-----	386,580	11,092	375,488	31,461	344,027	344,027	-----	-----	386,580	7,629	3,463	31,461	344,027	330,992	3.9
Oregon	5	5	-----	226,445	4,611	221,834	25,157	196,677	195,874	1	[20] 803	226,445	-----	6,487	23,831	196,127	186,193	5.3
Pennsylvania	4	4	-----	1,417,478	7,288	1,410,190	-----	1,410,190	1,410,190	-----	-----	1,417,478	-----	7,288	*56,699	1,353,491	1,225,894	10.4
Rhode Island	2	3	Apr. 22	122,334	826	121,508	2,979	118,529	118,529	-----	-----	122,334	-----	1,260	2,430	118,644	114,057	4.0
South Carolina	6	6	-----	187,024	-----	187,024	4,313	182,711	182,711	-----	-----	187,024	-----	4,313	*9,351	173,360	152,959	13.3
South Dakota	4	4	-----	116,797	6,917	109,880	2,145	107,735	97,347	(21)	10,388	116,797	4,672	2,245	12,533	97,347	96,750	.6
Tennessee	7	7	-----	265,142	14,345	250,797	-----	250,797	250,797	-----	-----	265,142	2,528	11,817	*13,257	237,540	239,348	−.8
Texas	4	4	-----	1,217,241	23,086	1,194,155	152,692	1,041,463	1,041,463	-----	-----	1,217,241	12,070	21,981	141,727	1,041,463	958,958	8.6
Utah	4	4	-----	91,110	5,368	85,742	-----	85,742	85,742	-----	-----	91,110	2,648	2,720	*10,022	75,720	69,013	9.7
Vermont	4	4	-----	65,797	1,917	63,880	-----	63,880	63,880	-----	-----	66,442	*645	1,917	*1,993	61,887	55,880	10.7
Virginia	5	5	-----	342,626	-----	342,626	19,959	322,667	322,667	-----	-----	350,228	*1,722	*5,880	19,959	322,667	298,888	8.0
Washington	5	5	-----	336,280	5,397	330,883	25,801	305,082	305,082	-----	-----	336,280	-----	5,397	25,801	305,082	288,560	5.7
West Virginia	4	5	Apr. 1	196,236	-----	196,236	4,157	192,079	192,079	-----	-----	198,128	-----	*2,198	3,851	192,079	174,930	9.8
Wisconsin	4	4	-----	540,545	16,760	523,785	39,470	484,315	484,315	-----	-----	540,545	13,430	3,386	39,414	484,315	452,709	7.0
Wyoming	4	4	-----	64,027	1,935	62,092	-----	62,092	62,092	-----	-----	64,027	-----	1,935	*7,043	55,049	50,569	8.9
District of Columbia	2	2	-----	137,301	5,559	131,742	662	131,080	131,080	-----	-----	137,301	318	5,241	662	131,080	121,625	7.8
Total	Weighted average rate 3.91 cents.			21,208,977	650,594	20,558,383	1,109,279	19,449,104	19,329,788	-----	119,316	21,345,536	264,571	313,297	1,549,547	19,218,121	17,855,454	7.6

[1] Export sales and other amounts not representing consumption in State have been eliminated as far as possible. Exemptions and refunds in a few States include small amounts of such deductible items not reported separately. In cases where States failed to report amounts exempted from taxation the gross amount taxed is shown in this column.

[2] Includes allowances for evaporation and other losses, Federal use, other public use, and nonhighway use, where initial exemptions rather than refunds are made.

[3] In the following States percentage allowances are made to distributors for expense of tax collection: Georgia, 1 percent beginning Apr. 1, 1937; Illinois, not more than 2 percent; Nebraska, ½ to 1 percent; Nevada, 2 percent; New York, 1 percent; North Dakota, 1½ percent; Pennsylvania, 2 percent. In tabulating the net amount taxed these allowances have not been deducted.

[4] In States where rate changed during the year the gallons taxed at different rates were as follows: Minnesota, 110,343,000 at 3 cents, 322,800,000 at 4 cents; Nebraska, 202,938,000 at 5 cents, 19,512,000 at 4 cents; New York, 519,817,000 at 3 cents, 1,182,804,000 at 4 cents; Rhode Island, 31,331,000 at 2 cents, 87,198,000 at 3 cents; West Virginia, 41,070,000 at 4 cents, 151,009,000 at 5 cents.

[5] In these columns available data have been used to determine approximately the amount of highway use and other uses in all States. The approximate methods used are described in notes 6 to 9.

[6] In the case of States indicated by stars allowances for evaporation, spillage, and other losses have been estimated on the basis of the percentages allowed by law and other available data. Where no data are given there is no percentage allowance, although actual proved losses are generally deducted from the taxable amount.

[7] Includes amounts exempted or subject to refund because of sale to the Federal Government. In the case of States indicated by stars Federal use has been estimated on the basis of the number of Federal vehicles and other available data. Amounts exempted or subject to refund for State, county, and municipal use also included, as follows: Colorado, 6,192,000 gallons; Delaware, 607,000; Florida, 431,000; Iowa (estimate), 7,640,000, Michigan, 7,186,000; New Jersey, 9,535,000; New York, 45,537,000; Oregon, 1,876,000; Rhode Island, 434,000; Tennessee, 4,480,000; Texas, 10,100,000; West Virginia, 263,000. For other States no information was given. Totals as follows: Federal use, 219,016,000; other public use, 94,281,000.

[8] Includes amounts exempted, subject to refund, or taxed for nonhighway use. In the case of States indicated by stars (*) no exemptions or refunds are made for nonhighway use. Amounts have been estimated, as percentages of total consumption, by comparison with data on nonhighway use in adjacent States.

[9] 1936 data revised to conform with 1937 analysis. Differences between amounts in this column and corresponding amounts in table Motor Fuel Consumption, 1936, previously issued, are due to adjustment of refund data to approximate month of purchase, estimate of nonhighway use in nonrefund States, and minor adjustments.

[10] Within 300 feet of border, tax is reduced to that of adjacent State. Gallons taxed at 2 cents, 3,106,000; at 4 cents, 12,598,000.

[11] Motor fuel used in aviation.

[12] Includes 3,000 gallons of inferior grade of gasoline taxed at 6 cents; and 6,839,000 gallons taxed at 2 cents, representing evaporation or loss allowance under the 5-cent tax not allowed under the additional 2-cent tax, which is administered under a separate law. Additional loss allowance of 160,000 gallons was fully exempted from taxation.

[13] 3 cents per gallon refunded on nonhighway uses.

[14] 1 cent per gallon refunded on motor fuel used in vehicles licensed to operate exclusively in cities.

[15] 1½ cents per gallon refunded on motor fuel used in interstate aviation.

[16] 5 cents per gallon refunded on nonhighway uses.

[17] Tax was 5 cents Jan. 1 to Feb. 28; 4 cents Mar. 1 to 24; 5 cents Mar. 25 to Dec. 31.

[18] Amounts given do not include 68,390,000 gallons of liquid fuel (kerosene, fuel oil, etc.) taxed at 1 cent per gallon but not subject to the 3-cent tax on motor-vehicle fuel.

[19] 3 cents per gallon refunded on motor fuel used for nonhighway purposes.

[20] 4 cents per gallon refunded on motor fuel used in aviation.

[21] Refunds on nonhighway use were made at 2 cents per gallon Jan. 1 to June 30; and at 3 cents July 1 to Dec. 31. Full refund of 4 cents on all claims paid after Dec. 31, 1937

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF MAY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,253,020	$ 616,510	65.3	$ 5,480,902	$ 2,736,705	270.9	$ 3,084,950	$ 1,540,670	75.4	$ 5,556,443
Arizona	2,393,808	1,672,609	109.3	1,773,871	1,150,634	95.1	318,038	173,827	17.1	1,909,171
Arkansas	3,090,778	3,061,971	187.4	1,091,511	1,083,185	73.5	60,246	58,838	7.7	4,327,715
California	7,134,798	3,864,782	154.8	9,212,545	4,841,754	165.7	4,354,315	2,310,481	100.6	2,701,226
Colorado	3,179,438	1,741,332	124.9	2,413,756	1,324,514	79.3	486,120	260,476	9.6	3,333,875
Connecticut	817,358	406,276	9.3	445,750	212,589	3.1	1,384,690	676,415	14.4	1,506,510
Delaware	534,818	263,575	21.0	377,568	188,529	4.7	530,055	250,440	17.5	1,399,843
Florida	1,054,653	524,469	29.0	2,451,442	1,225,721	51.8	1,027,760	513,880	18.2	3,822,610
Georgia	2,377,634	1,149,465	126.7	6,582,343	3,291,172	317.0	2,786,540	1,393,270	146.6	5,969,864
Idaho	2,723,739	1,587,662	211.8	1,119,310	667,278	77.1	1,282,306	760,707	59.6	1,496,429
Illinois	11,678,019	5,700,751	342.2	8,450,689	4,147,773	171.5	4,563,700	2,281,850	116.9	4,007,173
Indiana	6,185,861	3,036,230	149.6	3,873,841	1,936,920	103.7	3,720,719	1,854,010	80.5	2,923,627
Iowa	7,134,059	3,365,417	234.2	6,057,640	2,804,456	177.7	1,981,400	815,542	70.6	2,309,328
Kansas	4,671,980	2,306,728	262.8	3,642,858	1,821,330	94.8	4,556,719	2,234,710	747.4	4,251,501
Kentucky	2,780,677	1,378,557	68.4	4,874,632	2,437,316	164.3	3,476,112	1,725,314	112.4	2,443,135
Louisiana	582,530	286,501	15.0	6,378,335	1,381,335	39.5	7,092,858	1,615,531	51.7	2,720,809
Maine	1,958,287	961,207	54.8	2,270,096	1,132,113	51.7	1,339,570	669,784	25.6	607,963
Maryland	1,048,330	524,145	15.0	1,696,594	846,534	27.4	1,315,360	628,127	12.8	2,114,449
Massachusetts	4,417,322	2,208,609	20.3	2,523,521	1,261,760	8.3	446,240	221,820	2.8	3,127,964
Michigan	7,094,121	3,338,946	173.8	6,427,415	3,108,612	130.8	444,850	221,800	9.8	3,849,406
Minnesota	6,625,480	3,272,065	341.5	4,362,993	2,170,147	194.1	1,450,898	714,347	54.6	3,949,874
Mississippi	3,127,801	1,548,263	157.8	5,291,290	2,603,840	230.5	3,476,270	997,935	144.6	3,680,064
Missouri	9,109,675	4,435,132	468.2	5,029,894	2,485,285	155.7	3,826,287	1,607,972	148.5	4,095,850
Montana	3,716,751	2,078,017	277.7	2,416,103	1,358,675	116.9	280,841	157,972	8.7	4,458,935
Nebraska	2,660,483	1,315,014	276.4	5,754,277	2,864,967	507.6	4,172,366	1,394,241	223.3	3,148,696
Nevada	2,491,341	2,051,622	132.5	911,844	790,273	75.3	566,129	490,934	23.3	1,791,000
New Hampshire	358,791	176,896	6.5	991,407	492,095	16.9	457,371	226,501	9.7	1,150,804
New Jersey	2,055,958	933,456	20.3	2,762,676	1,380,133	19.9	129,000	64,500		2,776,553
New Mexico	4,412,241	2,708,439	292.7	1,555,908	1,048,835	84.2	1,191,506	726,693	154.9	1,467,892
New York	14,167,589	6,394,190	250.7	13,800,694	6,819,950	235.4	3,715,748	1,820,424	62.6	5,117,185
North Carolina	5,222,986	2,607,430	387.0	6,459,353	3,038,072	262.0	1,287,860	632,710	98.3	3,784,614
North Dakota	1,114,612	1,036,055	186.5	2,505,131	2,426,901	155.2	493,650	445,458	56.0	3,836,905
Ohio	4,510,308	2,175,379	58.8	9,151,132	4,545,120	101.6	1,922,760	961,340	17.4	8,947,494
Oklahoma	3,728,885	1,940,482	186.7	3,744,831	1,939,510	136.6	3,072,895	1,617,593	143.7	4,088,441
Oregon	4,190,974	2,451,357	135.3	2,089,962	1,231,075	101.8	605,330	369,320	19.1	2,529,698
Pennsylvania	13,148,101	6,491,718	179.2	7,376,241	3,675,331	117.0	2,252,002	1,122,664	39.9	6,862,430
Rhode Island	923,039	443,128	8.1	1,184,060	592,030	16.7	111,197	55,598	1.1	1,223,518
South Carolina	3,762,555	1,590,753	260.9	5,504,881	2,348,743	247.9	1,135,704	518,986	58.2	2,069,431
South Dakota	2,223,721	1,257,452	228.5	3,045,772	1,687,300	276.6	1,818,710	1,013,580	123.4	3,619,406
Tennessee	1,989,348	990,687	71.3	4,853,574	2,426,287	145.1	1,058,200	444,060	40.1	5,299,764
Texas	14,755,214	7,286,292	967.8	11,153,440	5,530,407	626.4	4,851,647	2,314,721	288.1	9,207,058
Utah	1,429,642	999,693	136.8	939,560	672,620	52.7	835,970	592,804	86.0	1,750,068
Vermont	1,159,310	551,784	31.6	1,598,151	698,154	41.1	433,509	207,145	9.7	461,814
Virginia	4,432,577	2,168,083	172.4	4,717,748	2,357,287	151.4	2,285,526	1,142,763	81.7	1,703,944
Washington	2,363,913	1,229,813	76.0	4,022,187	2,111,524	54.2	1,418,729	728,000	37.2	1,381,197
West Virginia	1,494,901	760,504	42.7	1,575,227	1,045,523	46.7	854,187	543,102	23.5	2,701,952
Wisconsin	8,349,032	4,063,619	278.8	5,468,484	2,529,577	117.4	2,070,557	879,843	74.6	2,959,848
Wyoming	2,302,388	1,748,262	300.7	1,477,642	904,462	150.1	998,775	613,282	115.6	938,327
District of Columbia										
Hawaii	845,057	412,657	13.3	727,000	360,215	13.2	765,270	375,695	12.7	1,289,017
Puerto Rico				1,075,539	535,790	20.3	60,920	30,360	.2	668,225
TOTALS	199,283,303	103,113,984	8,322.3	198,691,620	100,270,358	6,578.4	91,852,362	43,018,035	3,853.9	157,339,045

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF MAY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grede Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grede Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 16,100	$ 16,100	1			$ 228,501	$ 228,427	4			$ 336,850	$ 336,850	4		6	$ 1,420,242
Arizona						4,718	4,718									625,495
Arkansas						251,180	250,023	6			28,459	28,459	1			1,480,287
California						1,234,430	1,233,855	2	4		144,157	144,157	1			2,322,197
Colorado	1,880	1,880			1	26,240	26,240	1			4,491	4,491			2	1,257,311
Connecticut																844,490
Delaware											55,000	56,000			15	437,750
Florida						10,616	10,616									1,395,181
Georgia						18,346	18,346									2,399,041
Idaho						100,167	99,994	1								722,876
Illinois						728,175	728,175	3	2		595,600	595,600	7		1	3,900,368
Indiana	165,790	165,790	2			671,090	566,813	4	5		549,060	466,600	2		86	1,379,293
Iowa	172,088	163,900	3	3	1	976,983	923,800	12	1	4	125,876	117,200	1		16	1,568,746
Kansas	6,290	6,290			3	471,660	471,660	10		3	436,095	436,095	6		3	1,665,048
Kentucky						145,000	145,000	1			167,924	167,924	1		31	1,501,121
Louisiana						23,648	23,648				122,838	122,830	2			1,430,593
Maine						140,057	140,057		1		117,610	117,610	2	1	1	433,536
Maryland						7,800	7,800				664,000	56,130	1			962,903
Massachusetts						70,420	70,420		1							1,999,051
Michigan						476,972	476,972	3			667,820	667,820	7	1	45	2,140,393
Minnesota	174,236	174,236	1		5	608,748	608,748	2	4	12	98,650	98,650		2		1,775,066
Mississippi						252,700	252,700	3								1,331,451
Missouri						236,070	236,070	4			149,140	149,140	2			2,640,043
Montana						258,180	253,298	3			404,026	404,026	3			667,147
Nebraska						210,137	210,137	10			147,756	147,756	3			1,406,517
Nevada	35,109	35,109		1		91,561	91,561		2		52,574	52,574	1			314,506
New Hampshire						65,482	65,175		1							428,575
New Jersey						210,005	204,779	2	1							1,765,478
New Mexico						122,441	122,441	4								729,050
New York						1,415,251	1,414,100	5	4	2	553,946	546,746	3	1		4,809,083
North Carolina						135,400	135,400		1		420,550	420,550	4			1,931,624
North Dakota						534,046	534,046	1	1							1,045,175
Ohio						32,120	32,120									4,197,048
Oklahoma						17,343	17,343									2,264,166
Oregon	178,890	178,890	3			293,683	293,683	1	1		32,021	31,372		1		650,276
Pennsylvania						181,628	166,422	1			100,000	100,000	1			5,466,113
Rhode Island						223,897	223,897	1								269,853
South Carolina						38,473	16,137		1		324,844	324,844	2	1	2	1,148,121
South Dakota	43,770	43,770			13	124,438	124,438	1	1	1	73,505	73,505	1		6	1,126,744
Tennessee						14,381	14,381									1,877,652
Texas	19,700	19,700	1			77,133	76,200	4			94,940	94,605	1			5,208,363
Utah	71,000	71,000	2		2	105,073	105,073	2			6,860	6,860			4	461,055
Vermont	12,028	12,027			4	213,108	208,108	6	1	1	26,450	22,170		1		251,445
Virginia	80,710	80,710	2	3		230,395	230,395	12	1		106,720	106,720		1	3	1,458,915
Washington	80,215	80,215	1	1		232,133	230,033	2	2		268,368	268,368	4		3	939,956
West Virginia						139,175	138,155	1			267,660	267,660	3			918,283
Wisconsin						875,054	874,880	7			163,203	136,907	2			1,461,722
Wyoming						144,884	144,884		2		14,460	14,460			6	517,386
District of Columbia	168,320	168,320		1												325,431
Hawaii											197,540	197,540	3	1		296,210
Puerto Rico						61,900	61,550	2			102,621	101,740	1			567,010
TOTALS	1,226,126	1,217,937	16	9	29	12,730,842	12,512,718	121	37	23	7,622,614	6,883,959	69	10	230	78,135,386

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF MAY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 209,700	$ 104,850	11.9	$ 219,500	$ 109,650	11.9	$ 838,472
Arizona	$ 33,909	$ 23,138	2.4	204,974	117,189	11.4	157,793	107,507	14.7	475,353
Arkansas				13,126	6,563					857,545
California	105,952	61,502	30.2	454,226	261,987	30.1	299,350	173,746	31.5	1,421,463
Colorado				394,396	218,488	19.3				703,080
Connecticut							12,960	6,465	.2	311,813
Delaware										246,875
Florida				20,122	10,061					664,791
Georgia	6,000	3,000	1.8	232,598	116,299	20.2	350,580	175,060	52.1	983,267
Idaho	233,905	114,838	33.7	310,743	144,404	17.6	52,683	26,341	4.2	336,190
Illinois	36,000	18,000	3.3	804,632	402,316	57.6	1,373,900	684,250	108.6	962,249
Indiana				216,800	73,150	22.6	1,015,633	449,003	100.6	720,285
Iowa										1,298,449
Kansas	19,510	9,755	24.0	49,234	24,617	.9	131,450	65,725	6.7	1,231,364
Kentucky	246,502	122,573	105.1	532,214	148,425	50.6	630,565	179,286	66.3	479,393
Louisiana				21,670	10,835		117,099	54,696	11.1	658,605
Maine	58,904	29,140	4.1	311,636	155,818	20.0	120,800	60,400	7.6	197,048
Maryland				6,264	3,132					409,344
Massachusetts				5,300	2,650					696,530
Michigan				23,362	11,681					1,523,858
Minnesota				265,714	130,950	6.4	141,164	50,477	39.1	1,198,153
Mississippi										888,927
Missouri	954,315	458,965	242.3	275,810	136,605	39.1	231,700	96,235	16.9	838,759
Montana				13,983	7,865					1,027,170
Nebraska	40,394	20,197	1.1	276,210	138,105	42.6	378,686	189,343	51.5	696,463
Nevada	177,826	153,860	26.5	190,948	165,308	15.5	82,224	67,423	18.7	257,920
New Hampshire				101,176	50,152	1.7	61,884	30,904	1.5	165,819
New Jersey							42,310	17,500	.1	655,273
New Mexico				426,365	260,037	27.5	115,029	70,156	3.2	476,579
New York	119,825	58,962	12.8	2,212,860	1,106,430	145.7	188,800	94,400	5.5	1,212,981
North Carolina	132,350	66,062	33.1	618,940	309,470	76.5	169,000	78,980	13.4	729,045
North Dakota							31,450	16,844	5.5	770,408
Ohio				184,400	92,200	3.8				1,739,441
Oklahoma				16,888	8,986		199,160	95,932	26.0	1,079,997
Oregon	111,890	62,115	33.8	84,422	50,532	10.7	420,020	245,120	45.6	470,390
Pennsylvania	33,202	16,601	4.9	607,217	292,506	38.6	1,038,548	498,128	72.3	1,337,983
Rhode Island	25,162	10,355	.3	118,988	59,494	3.6	89,590	44,795	2.6	132,231
South Carolina				408,398	169,862	40.4	336,290	143,354	41.1	365,464
South Dakota				11,300	6,250					816,436
Tennessee				184,806	92,403	7.8	71,520	35,760	4.8	931,327
Texas	22,300	11,150	9.7	491,768	240,645	102.8	1,450,918	658,772	200.2	2,238,503
Utah	16,812	11,750	2.1	220,770	134,610	21.9	191,065	91,205	13.7	334,151
Vermont	76,012	37,335	4.3	200,056	87,853	11.6				121,687
Virginia	40,210	20,105	10.2	319,168	157,369	42.2	320,460	160,230	7.2	581,945
Washington	117,776	61,874	18.0	244,960	128,778	15.4	265,697	139,800	22.9	461,391
West Virginia				208,000	104,000	16.5	82,300	41,150	4.5	404,524
Wisconsin	30,710	15,250	6.7	225,510	106,945	1.7	411,331	176,651	22.6	928,632
Wyoming	75,809	46,849	7.2	363,920	224,860	39.1	14,500	8,960	4.7	351,373
District of Columbia										
Hawaii							54,010	27,005	2.4	219,870
Puerto Rico				217,720	108,860	11.3	31,930	15,865	2.4	122,150
TOTALS	2,715,275	1,433,376	617.6	12,301,294	6,183,540	984.6	10,901,899	5,187,118	1,043.9	36,570,966

U. S. GOVERNMENT PRINTING OFFICE 1938

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 5 JULY 1938

A SECTION OF TWO-LANE HIGHWAY IN MISSOURI.

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the
UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
D. M. BEACH, *Editor*

Volume 19, No. 5 July 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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ACCIDENTS ON THE ROAD

BASES OF STATISTICS AND THEIR UNIFICATION; DETERMINATION OF CAUSES OF ACCIDENTS AND MEANS FOR DIMINISHING THEM [1]

By **SIDNEY J. WILLIAMS**, Director, Public Safety Division, National Safety Council

THE HIGHWAY ACCIDENT PROBLEM, like any other, demands searching diagnosis before treatment may be attempted. Our chief diagnostic instrument in this case is statistics. They provide the information by which we can weigh each element to determine its relative importance and decide where and how to apply remedies.

Fatality statistics are collected in the United States mostly by departments of the various States, usually the health department or the body administering the motor-vehicle laws. This system is facilitated by laws, in force in 35 of the 48 States and the District of Columbia, that require drivers involved in accidents resulting in injury or death to report them to some enforcement or administrative agency. In 31 of these States, property damage accidents likewise must be reported.

The Federal Bureau of the Census compiles the Nation-wide figures in connection with its tabulation of general vital statistics. Its annual motor-vehicle fatalities statistics are the ultimate authority.

The National Safety Council is constantly compiling monthly and annual statistics from States and cities and shortly after the close of each year publishes its statistical summary, Accident Facts. Since this publication is issued before the Census Bureau figures are released, it is necessary to estimate the total fatalities and other figures which this total governs.

These estimates have been exceedingly close from year to year and should tend to be more so in the future because a growing number of city and State departments are working to improve their accident reporting systems, thereby reducing the element of guess to a minimum. The increase in accident reporting activity is being furthered by the Council both by personal visits of staff members to reporting agencies and in consultations by mail.

Regarding the need for full reports of accidents, Mr. C. H. Purcell, State Highway Engineer of California, says: "The one principal source from which we may obtain information * * * is the individual accident report. Too much emphasis cannot be placed upon its importance. Great care should be used in the preparation of these report forms, with a clear understanding at all times of what use is later to be made of their contents. What is actually recorded on the form and not what the designer of the form may have expected to be recorded—is all that is of any value. For this reason only disappointment can follow if due consideration has not been given to the type of person upon whom we must depend for the common, standard accident report. Only by the accumulation of thousands of such reports can any sufficiently broad statistical base be built up.

" * * * The common, or standard, accident report will provide pertinent data as to the apparent physical condition of the persons involved and also as to their actions, but, except in the most obvious cases, will of course contain no direct reference to mental condition.

" * * * While complete statistics on the persons, the machines and the roads, are all basically essential, there still remains the one elemental factor without which there can be no meaning to any study of highway accidents. This is the amount and character of traffic. The number of accidents, judged solely on the basis of drivers, cars, and miles of road, may appear either excessive or inconsequential, but there can be no real basis for judgment until we know what kind of, and how much travel was involved during the period in which these accidents occurred."

Since 1924, the National Safety Council has sponsored the Standard Accident Reporting System not only as a means of presenting statistics uniformly, but also of bringing out those details which are necessary in complete diagnosis. Approximately 30 of the 48 States and some 300 cities now use the Standard System.

Another effort toward uniform accident reporting and wider coverage is being made by the Joint Committee on Uniform Accident Statistics, sponsored by the National Safety Council and the American Association of Motor Vehicle Administrators. The committee represents leading governmental agencies dealing with statistics who are cooperating to obtain similar definitions for the various types of vehicle accidents, and generally to improve the collection and compilation of figures so that those who are trying to reduce accidents will have the fullest information.

Concerning the need for complete statistics, and the possibilities they open, Mr. A. A. Anderson, Manager of the Highways and Municipal Bureau of the Portland Cement Association, contributes the following:

> Until recently no concerted action has been taken to obtain complete information regarding all factors contributing to highway accidents. Previous information has generally been confined to recording the few facts necessary to establish the responsibility for the accidents, and such general data as location, time, and weather.
>
> The inadequacy of such information is apparent. There is also a lack of uniformity in methods of reporting but with greater emphasis on the need of complete accident records, undoubtedly a uniform report will be developed.
>
> Many States now require, by law, a detailed report of all highway accidents and it is expected that greater light will soon be thrown on the real factors affecting conditions which cause accidents.
>
> Just as facts are required for proper planning of the highway system and structural design of the road, so are accident facts necessary before highway safety can be built into the highways and existing defects corrected.
>
> Basic information is needed from original accident reports. This should be collected and compiled in a uniform manner. It is essential that such information include data which will permit:
>
> a. Listing accidents on a vehicle-mile basis for specific highways (day and night).
> b. A rating to be made of the physical safety of each highway in relation to conditions causing accidents.
>
> This will mean mandatory provisions by the State or local government for reporting all details incident to highway accidents. Many States have already taken such steps.
>
> When all contributing factors are obtained it will be possible to develop an accident expectancy formula based on traffic volume and the physical rating of the highway. When such a formula is developed it will enable traffic engineers to:
>
> a. Predict the number of accidents on a highway having a given traffic and known physical rating.

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[1] Report for the United States to the Eighth International Road Congress held at The Hague, June 19–July 2, 1938.

b. Determine the accident reduction which would follow if the physical hazards were eliminated.
c. Estimate the expenditure which could be justified to remove physical hazards.

Every road has its own characteristics and by studying these along with the detailed accident reports, then without doubt the causes of accidents can be segregated and evaluated.

To get an idea of the size of the problem confronting us, it is only necessary to review the annual motor-vehicle fatality totals since 1913. These show clearly how the motor car is claiming more and more attention in accident prevention (table 1).

TABLE 1.—*Accident fatality totals*

Year	Number of motor-vehicle fatalities	Number of fatalities resulting from all accidents	Ratio of motor-vehicle to all accident fatalities
			Percent
1936	[1] 37, 800	[1] 111, 000	34
1935	36, 369	99, 967	36
1934	36, 101	101, 139	36
1933	31, 363	91, 087	34
1932	29, 451	89, 167	33
1927	25, 796	92, 874	28
1922	15, 326	76, 420	20
1917	10, 235	90, 116	11
1913	4, 227	82, 460	5

[1] Estimated.

This rising importance is shown also by the fact that in the United States there are only 4.55 people per vehicle of all types and 5.31 persons for each passenger automobile (1936).

Other facts that come to light in studying accident causes are shown in a general way by the following article, What Causes Traffic Accidents? published in the 1937 edition of Accident Facts:

> No one can say exactly how many motor-vehicle accidents in the United States are due to particular causes, because few accidents are investigated carefully enough to determine exactly what was the cause, and because most accidents have not one but a combination of several causes.
>
> *Highway defects.*—Starting with the less important causes, we know that many highways are not yet as safe as they could be made. They are too narrow, curves are too sharp, signs and signals are not standard, and so on. These, when combined with inattention or ignorance on the part of the driver, often result in accidents. The careful, skillful driver, however, rarely has an accident even on a defective highway.
>
> *Vehicular defects.*—Defects of the vehicle itself are estimated to cause or help to cause at least 15 percent of the accidents. The most important vehicular defects are defective brakes and deficient or glaring headlights. As with unsafe highways, however, the careful, skillful driver can generally avoid accidents, even if his vehicle is somewhat defective.
>
> *The pedestrian.*—Pedestrians act unsafely under many circumstances; children play in the street, * * * adults take foolish chances by crossing a street in mid-block, or against the signal, or when intoxicated—with disastrous results.
>
> *The driver.*—The driver is the most important element in our traffic accident situation. Some drivers have accidents because they are actually defective in mind or body * * *.
>
> A much greater number have accidents because they do not know how to drive. They have never learned proper methods of making turns, backing, or signaling; they do not observe right-of-way rules, stop signs, or traffic signals, to say nothing of their obligation toward pedestrians and other highway users.
>
> The greatest number of drivers who have accidents, however, are neither physically defective nor ignorant of traffic rules. They don't really want to have an accident, but neither do they sufficiently want not to have one. They take chances by going too fast; they pass other cars on hills and curves; they cross railroad tracks without looking; they do not slow down at intersections; they expect other drivers and pedestrians to get out of their way regardless of traffic conditions.
>
> *The remedy.*—After all, it is not so important that we cannot determine exactly what percentage of accidents is due to any one of these complex causes. We are interested in eliminating the accidents, and these causes tell us how to proceed. First, we must improve our highways and keep our vehicles in good condition. Second, and more important, we must teach drivers and pedestrians to realize the accident possibilities of all their actions on streets and highways and to act accordingly.

In discussing accidents chargeable to the driver, Dr. H. C. Dickinson, National Bureau of Standards, says:

> It seems that highway accidents for which drivers are responsible might be represented in two classes between which classes, of course, there is no rigid line of separation but which still possess important differences.
>
> Suppose we picture two classes of drivers which may be called good and bad for lack of a better distinction.
>
> The question which the writer wants to ask in all seriousness is: "What percentage of our accidents is due to drivers who individually would be classed as good drivers, drivers who do not customarily permit themselves to do the things which they know have caused accidents?"
>
> There seems to be no way of making even an intelligent guess as to what percentage of accidents is due to good drivers, mainly because we have no means as yet of finding out who are good and who are bad drivers.
>
> About all that can be said as to the relative number of accidents to good drivers as compared to bad drivers is that both are of parallel importance, either of them may be the more important.

Dr. Dickinson refers to what he terms an "accidental accident" as one in which all drivers involved are "good" drivers and explains that such an accident must result from a momentary lapse of the good qualities of one or more drivers.

This type, he says:

> * * * cannot be greatly reduced by punishment or enforcement. The good drivers cannot be improved by exhortation or by instilling fear because they already are doing the best they know how. * * * Confusing signs, bad visibility, curves which deceive the eye * * * all invite accidents on the part of the best and most careful driver.
>
> A more fruitful source of good-driver accidents may be the confusion of the regulations themselves. Rules of the road that ambiguously prescribe the right thing to do cause confusion and errors of judgment even among the best of drivers.
>
> One point about the class of good-driver accidents should be kept in mind. These drivers will differ greatly in their liability to accident. Equally well-trained, experienced, and conscientious persons in every line of activity differ in this way.
>
> The bad drivers on the other hand have received most of the attention. In their numbers they certainly are vastly in the minority, and they may be so even in respect to the number of accidents in which they are involved. Accidents that are due to this group presumably can be reduced by legal restraint and enforcement. They might be reduced more effectively if the minority who take chances would be identified and educated or disciplined.
>
> We need more information as to the relative importance of accidents which can be avoided and those which practically can not, if appropriate measures are to be applied to reduce highway accidents.
>
> If we are to greatly improve the situation, driving practices and rules of the road must be simplified and clarified so that they will subject the good drivers to a minimum of distraction from their normal safe behavior. We must find some means of catching and educating or punishing those who are really bad drivers, and we should get them before they cause an accident.

Before leaving the subject of accident causes, it may be interesting to present some of the significant circumstances which research has revealed as surrounding accidents in the United States.

For instance, it has been found that four out of five accidents occur on dry roads in clear weather and while the car is traveling straight ahead. Although only one-third of our traffic moves on the highways at night, two-thirds of the motor vehicle fatalities occur during the hours of darkness. This is explained by the fact that the total accident rate on a mileage basis during the night is much higher and that a higher percentage of all the accidents occurring at night result in fatalities.

More accidents happen on Sunday than any other day. Five p. m. is the most dangerous hour and 4 a. m. shows the lowest number of mishaps. A consensus of 26 State accident summaries for 1936 shows the 7 most important improper acts to be:

1. Exceeding the speed limit or driving too fast for conditions.
2. Driving on the wrong side of the road.
3. Disregarded stop signs and signals.
4. Improperly failing to yield the right-of-way at intersections.
5. Attempting to force entrance between moving vehicles to avoid collision with an oncoming vehicle.
6. Attempting to pass on a curve or hill a vehicle moving in the same direction.
7. Failing to signal intention to stop or turn.

Since repeal of the Federal liquor prohibition law a few years ago, a great deal of attention has been centered on the intoxicated motorist. Accident Facts makes the following comment:

> During 1936, the reported percentages in 26 States of "had been drinking" or "intoxicated" participants in fatal accidents averaged 7 percent for drivers and 11 percent for pedestrians. Cities reported an average of 8 percent for fatal accident drivers and 10 percent for pedestrians killed. That these percentages understate by an unknown amount the true situation is the belief of most traffic authorities. How high is the true percentage? That is the question.

A later paragraph in the same article says:

> The reported percentages of "alcohol accidents" have been increasing for the past several years with the exception of1936 in which year the percentage of drivers who "had been drinking" remained at 7 percent, the same as in 1935; only 6 percent were so recorded in 1934; and 5 percent in 1933. "Had been drinking" pedestrians were reported in only 9 percent of the 1935 fatal accidents, 8 percent in 1934, and 6 percent of the 1933 fatal accidents, compared with the 11 percent shown for 1936. Drinking drivers and pedestrians are playing an increasing part in the accident problem.

The discovery of accident causes is a challenge to safety agencies to start remedial measures. Accident reduction programs are carried on by a great many different organizations, each taking that part of the task for which it is best equipped.

Serving as a national clearing house for safety information, both public and industrial, is the National Safety Council. We shall, for the sake of brevity, emphasize its work in public safety.

The chief activity of this division is preventing street and highway accidents, although it also carries on home-safety and child-education programs. Membership comprises city, State, and national governmental departments, local safety councils, chambers of commerce, automobile clubs and other civic organizations, insurance, transportation, public utility, and other industrial companies. The Council is supported by the dues of these members and, thus, is nonpolitical and nonpartisan. It is in no way controlled by any branch of Government. Its services consist of technical advice and promotional activities, both by letters and personal visits as well as booklets and posters covering every phase of the problem.

The Council also conducts the National Traffic Safety Contest in which 1,101 cities compete for annual awards based on safety activities and reduction of motor-vehicle fatalities. Cities compete in their own population groups with a prize for the winner in each group and grand prize covering all groups. Forty-two States also are entered and are graded roughly on the same basis as cities.

Within the past few years, the automobile industry has taken an important part in accident prevention. The Automobile Manufacturers' Association has established the Automotive Safety Foundation, the main function of which is the giving of financial support to organizations working directly in the interest of safety. These organizations include the National Safety Council, International Association of Chiefs of Police, Harvard and Northwestern Universities, General Federation of Women's Clubs, National Congress of Parents and Teachers, and others.

Motor clubs and their associations, such as the American Automobile Association, are carrying on an active public safety program comprising such things as driving and safety courses for schools, dissemination of safe driving literature, and the support of safety legislation.

Insurance companies, both individually and through their organizations, are spending large sums to educate the public in the fundamentals of safety. They distribute booklets and other information as well as providing specific services to their clients. Operators of motor-vehicle fleets, for instance, can obtain the advice of their insurance company's fleet safety engineers.

Public safety is largely the responsibility of public officials who have authority to put the various measures into effect. The excellent manner in which many groups of officials have assumed this responsibility is demonstrated by the American Association of State Highway Officials. For several years it has actively advocated the standardization of traffic signs, signals, and markings as well as the incorporation of safety features in road construction. The American Association of Motor Vehicle Administrators is another safety-minded organization which has promoted uniformity of traffic laws and their administration.

The International Association of Chiefs of Police maintains a Safety Division with a full time director and a staff of assistants. The Safety Division conducts a service by which it trains police squads in scientific accident investigation. It also helps sponsor traffic officers' training schools in which working traffic officers are schooled in the latest developments of their job.

Although motor-vehicle laws are made and enforced primarily by the States, the Federal Government is attacking the traffic safety problem on many fronts. The Bureau of Public Roads administers Federal financial aid to State highway construction and thereby exerts a powerful influence in seeing that highway construction and alteration include the best safety principles. The Bureau has recently completed its Report to Congress on Study and Research of Traffic Conditions and Measures for Their Improvement. This report is a study of vehicle death rates, safety legislation, accident causes and suggested "cures" and is, in fact, a complete investigation of the problem.

The Federal Government does administer directly the regulation of trucks operating in interstate commerce through the Interstate Commerce Commission. It specifies the maximum size, weight, and loads of such vehicles, the safety equipment (such as distinguishing lights) each truck must carry, and similar safety measures.

Transit companies, such as tram and bus lines, are working to solve two big problems: One is the operation of their vehicles in such way as to cause a minimum of traffic congestion. This includes the elimination of collisions and the development of means for taking on and discharging passengers without obstructing other vehicles. The other is the reduction of accidents in which pedestrians and passengers are hurt either by being struck by the vehicle or falling while entering, riding, or alighting.

Such groups as the General Federation of Women's Clubs and the National Federation of Business and Professional Women's Clubs also cooperate in broad safety programs. The General Federation encourages its local affiliates to sponsor a safety program in their States and communities, including the support of safety legislation.

The promotion of general public safety now is carried on in 37 States by organizations of citizens and officials comprising what we shall call here State safety councils, although they may bear a variety of names, such as traffic commissions and safety committees.

A typical State safety council consists of a board of directors comprising the president, vice president, and representatives of all bodies concerned. These include State officials, such as the Governor, highway commissioner, motor-vehicle administrator, chief of the highway patrol, chairman of the public utilities and industrial commissions, commissioner of education, and others. There are also representatives of business and industrial groups who have an active interest in safety and civic leaders representing local, civic, patriotic, and other interested organizations. Another group represented on the board of directors is the chairman of each recognized county and local safety council in the State. Responsible to the board of directors is an executive committee made up of council officers and the chairmen of standing committees handling such work as legislation, education, statistics, engineering, and enforcement. There is also an executive who devotes his full time to handling council affairs. These councils may be supported financially by the State or by private subscriptions.

The accompanying chart, "Highway Safety Program for States," outlines the seven points necessary in a comprehensive accident reduction campaign. Not every State having a safety organization has all of these seven program activities in effect, but the States that have the best accident reduction records have more of them than the States with poorer records.

The above described program is not intended for cities or local communities. A plan of action suitable for these smaller and more compact units may include all or any part of the following:

1. *Statistics.*—Endeavor to introduce modern accident-reporting systems and convenient record systems; compile and interpret monthly accidents totals and release them through newspapers; study and use the statistics as a basis for suggested remedies directed along lines of engineering, education, and enforcement.
2. *Traffic engineering.*—Work for the revision of local ordinances to conform with the model advocated by the National Conference on Street and Highway Safety; promote the installation of a traffic engineering department in the city government; conduct independent studies of traffic conditions; and support all efforts toward the standardization of signs, signals, and markings.
3. *Public education.*—Develop a regular publicity program to keep the public informed of the council's work; have the city entered in the National Traffic Safety Contest; conduct safety meetings and special campaigns for safe driving.
4. *School and child safety.*—Encourage safety instruction in schools, to be supervised by a responsible official; organize school safety patrols and junior safety councils; and carry out a program of playground safety.
5. *Commercial vehicle safety.*—Conduct an interfleet safety contest, holding regular meetings of superintendents and drivers to promote the safe operation of commercial vehicles.
6. *Traffic law enforcement.*—In cooperation with the police and the courts, urge the establishment of a traffic division and accident investigation squads as part of the police force; help eliminate official misconduct which allows violators to escape punishment; and encourage the separation of traffic courts from those handling other kinds of cases.

Safety work is done in the local communities by county or city safety councils. If the towns in a county are all small, a single county safety council is most effective. For the larger cities, on the other hand, separate city councils function better.

A typical community safety council should include persons with an economic interest in safety, officials with a public responsibility, and public spirited citizens. The organizations should be headed by a director who may devote all or only a portion of his time to the work depending upon the size of the council and the fullness of its program.

Legislation.—Uniform motor-vehicle laws for all States and communities is the goal of most safety organizations. In addition to uniformity, these organizations also are striving for the adoption of the most modern legislation, as represented by the several proposed acts sponsored by the National Conference on Street and Highway Safety, collectively known as the Uniform Vehicle Code. Special emphasis is placed upon Acts II, the "Uniform Operators' and Chauffeurs' License Act" and V, "Uniform Act Regulating Traffic on the Highways." The three other measures in the Code deal with registration of vehicles, civil liability, and financial responsibility; hence, they are not concerned directly with accident reduction.

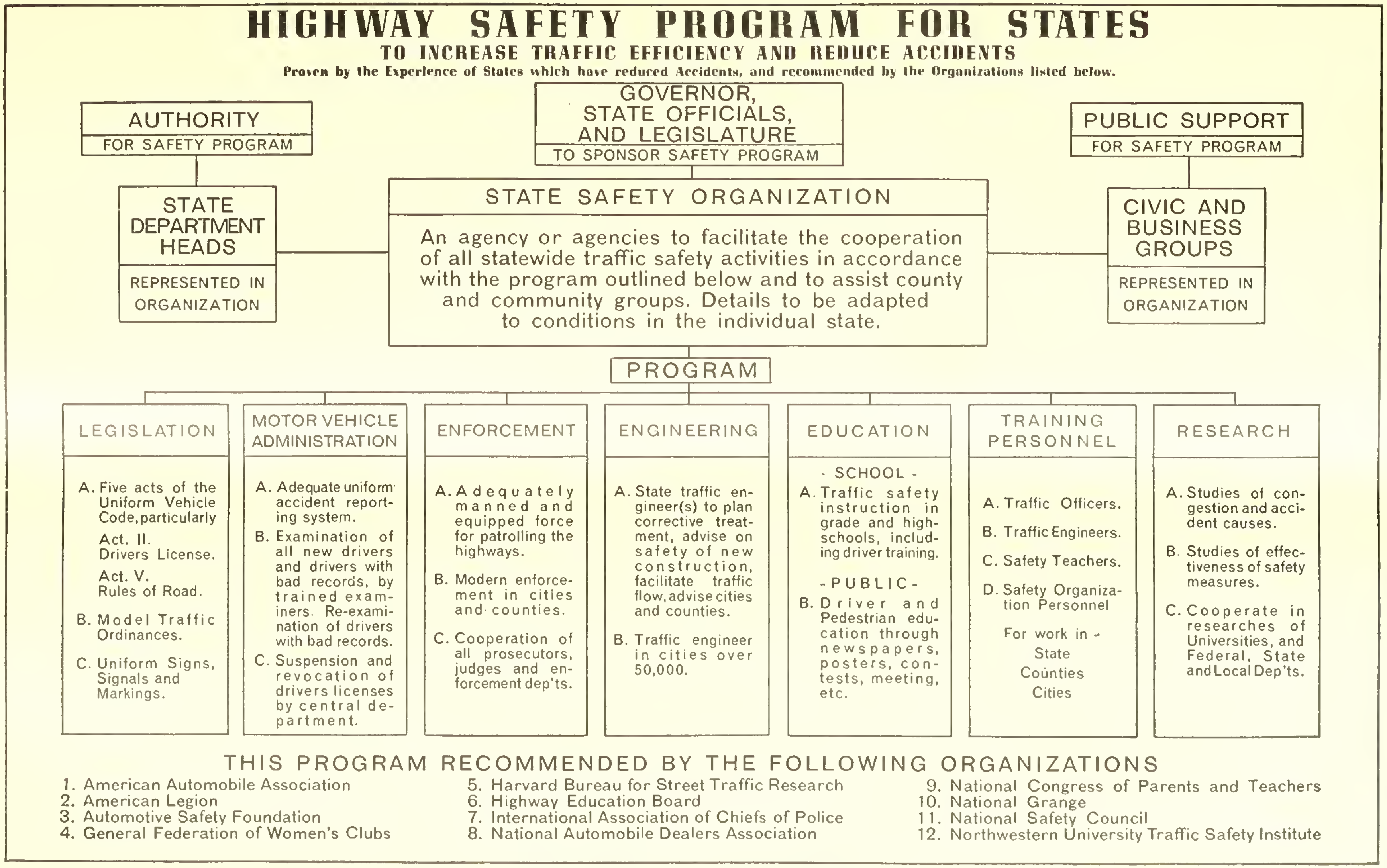
Cities are urged to adopt the Model Traffic Ordinances so that their laws also will be uniform and of an approved type.

The third portion of the legislative program involves the installation of uniform signs, signals, and markings. The different types of warning and directional signs, for instance, can be identified, if they are standard, not only by the legend they bear, but by their shape and coloring. The value of this to a motorist in strange territory is obvious.

Motor-vehicle administration.—It is, of course, of no value to have good laws if they are not properly administered. The State program under this heading includes, first of all, an adequate uniform accident reporting system. Next is the alert administration of the uniform drivers' license law which includes examination of all new applicants for permits and the reexamination of drivers with bad records.

Since a drivers' license law is, in effect, a method of eliminating dangerous drivers, proper machinery for revoking and suspending the permits of such drivers is an important function. Experience has shown that this works best when controlled by some department of the State government rather than by local courts.

Enforcement.—The first requisite of enforcement is sufficient personnel. This includes not only an adequate, well-equipped highway patrol for the State, but similarly well organized forces in the counties and cities. The police and judges must cooperate if vio-



HIGHWAY SAFETY PROGRAM FOR STATES

TO INCREASE TRAFFIC EFFICIENCY AND REDUCE ACCIDENTS

Proven by the Experience of States which have reduced Accidents, and recommended by the Organizations listed below.

GOVERNOR, STATE OFFICIALS, AND LEGISLATURE
TO SPONSOR SAFETY PROGRAM

AUTHORITY
FOR SAFETY PROGRAM

PUBLIC SUPPORT
FOR SAFETY PROGRAM

STATE DEPARTMENT HEADS
REPRESENTED IN ORGANIZATION

STATE SAFETY ORGANIZATION

An agency or agencies to facilitate the cooperation of all statewide traffic safety activities in accordance with the program outlined below and to assist county and community groups. Details to be adapted to conditions in the individual state.

CIVIC AND BUSINESS GROUPS
REPRESENTED IN ORGANIZATION

PROGRAM

LEGISLATION

A. Five acts of the Uniform Vehicle Code, particularly
   Act. II. Drivers License.
   Act. V. Rules of Road.
B. Model Traffic Ordinances.
C. Uniform Signs, Signals and Markings.

MOTOR VEHICLE ADMINISTRATION

A. Adequate uniform accident reporting system.
B. Examination of all new drivers and drivers with bad records, by trained examiners. Re-examination of drivers with bad records.
C. Suspension and revocation of drivers licenses by central department.

ENFORCEMENT

A. Adequately manned and equipped force for patrolling the highways.
B. Modern enforcement in cities and counties.
C. Cooperation of all prosecutors, judges and enforcement dep'ts.

ENGINEERING

A. State traffic engineer(s) to plan corrective treatment, advise on safety of new construction, facilitate traffic flow, advise cities and counties.
B. Traffic engineer in cities over 50,000.

EDUCATION

- SCHOOL -
A. Traffic safety instruction in grade and high-schools, including driver training.

- PUBLIC -
B. Driver and Pedestrian education through newspapers, posters, contests, meeting, etc.

TRAINING PERSONNEL

A. Traffic Officers.
B. Traffic Engineers.
C. Safety Teachers.
D. Safety Organization Personnel

For work in -
State
Counties
Cities

RESEARCH

A. Studies of congestion and accident causes.
B. Studies of effectiveness of safety measures.
C. Cooperate in researches of Universities, and Federal, State and Local Dep'ts.

THIS PROGRAM RECOMMENDED BY THE FOLLOWING ORGANIZATIONS

1. American Automobile Association
2. American Legion
3. Automotive Safety Foundation
4. General Federation of Women's Clubs
5. Harvard Bureau for Street Traffic Research
6. Highway Education Board
7. International Association of Chiefs of Police
8. National Automobile Dealers Association
9. National Congress of Parents and Teachers
10. National Grange
11. National Safety Council
12. Northwestern University Traffic Safety Institute

Highway Safety Program for States.

lators of the vehicle laws are to be punished. For the officer this means the gathering of sufficient and convincing evidence and the ability to present it properly in court. For the judge it means a fair attitude, no mistaken ideas of leniency or persecution, and a realization of the seriousness of the accident problem. Honesty and freedom from political pressure are vital in both officials.

Engineering.—A good traffic engineering bureau in the State government can do two things. It can put the best and most modern engineering treatment into effect on highways controlled by the State, and it can advise cities and counties in their problems. The State also should do whatever it can to induce cities over 50,000 population to have their own traffic engineers.

Education.—Inclusion of definite safety courses in school curricula is spreading in the United States. This should be furthered in the State program, and should provide for actual courses in driving automobiles. Public education also is an important factor which should be carried on through the newspapers and radio, in much the same way as it is carried on by the community safety councils.

Training personnel.—The promotion of public safety has brought into existence a new profession for which the training of leaders and personnel is necessary. These include traffic officers, who now attend the half dozen or so training schools held in various parts of the United States, traffic engineers, and general safety organizers, whose job it is to stimulate interest and to carry these safety programs through to a worth-while conclusion.

Research.—This report has discussed earlier several of the accident causes and significant circumstances of accidents. Constant research is necessary to discover more and more of these facts and to hunt for any new ideas that may make remedies more effective. Research is necessary also to determine the relative effectiveness of new safety measures as they are proposed and assign each to the place in the whole safety scheme in which it will do the most good.

Results of organized effort.—That a well-rounded safety program will work, that it will reduce accidents, is proved conclusively by several American cities. Table 2 shows the accident experience of these cities which applied the recommended programs.

TABLE 2.—*Accident records of cities that applied recommended safety programs*

City	Population	Average death rate 5 years ending 1931	Average death rate 5 years ending 1936	Percent change
New York, N. Y.	6,930,400	16.4	13.7	−16.5
Baltimore, Md.	804,800	18.5	16.7	−9.7
Milwaukee, Wis.	578,200	17.2	12.1	−29.7
Rochester, N. Y.	328,100	12.8	12.3	−3.9
Dallas, Tex.	260,500	21.2	16.4	−22.6
Providence, R. I.	253,000	13.9	9.8	−29.5
Syracuse, N. Y.	209,300	16.4	9.8	−40.2
Wilmington, Del.	106,600	15.0	13.5	−10.0
Allentown, Pa.	92,600	14.5	14.1	−2.8
Evanston, Ill.	63,300	13.7	7.7	−43.8

Figures for cities which did not employ effective safety programs reveal much higher death rates, especially during the 5-year period ending with 1936. Not one of the cities listed above had a death rate as high as the average of all cities in its own population group.

This demonstrates that accidents can be reduced with intelligent leadership, cooperation of all interested groups including the general public, and a comprehensive program attacking the problem along lines of engineering, education, and enforcement. We certainly do not know all there is to know about accidents, but we do know their chief causes and something of how to remedy them. The process, then, is one of recruiting an army of officials and citizens to help, and of educating everyone to his own responsibility whether it be for walking safely across the street or for establishing a State-wide program.

The United States is awake to the need for safety as it never has been in the past. The big task of the moment is to coordinate all the good work that is being done and to guide enthusiasm into channels by which it will achieve the best results.

ACKNOWLEDGMENTS

The Principal Reporter acknowledges the very valuable papers contributed by Mr. A. A. Anderson, Manager of the Highways and Municipal Bureau of the Portland Cement Association; Dr. H. C. Dickinson, of the National Bureau of Standards; and Mr. C. H. Purcell, State Highway Engineer of California.

Mr. Anderson's shorter paper is quoted in this report in its entirety and excerpts have been taken as indicated from the papers by Dr. Dickinson and Mr. Purcell.

SEGREGATION OF THE VARIOUS CLASSES OF TRAFFIC ON THE HIGHWAY

Reported by R. E. TOMS, Chief, Division of Design, United States Bureau of Public Roads

Introduction.—The increase in the number of motor vehicles in the United States from 640,000 to 28,000,000 over a period of 25 years, together with steady increases in the mileage traveled by each vehicle and in the speed of traffic, has made necessary the development and construction of many ingenious designs for traffic movement, segregation, and safety.

Carriageways.—The principal items controlling the distribution of vehicular traffic on the road are traffic density, character of traffic and assumed design speed. The American Association of State Highway Officials is considering the classification of all highways in terms of these essential factors to indicate readily the services which may be expected of a highway in each class. Each classification is indicated therein by two figures and a letter. For example, the classification 1000 M 50 indicates a highway on which a mixed vehicular traffic of 1,000 vehicles per hour can be accommodated at an assumed design speed of 50 miles per hour; 600 T 30 indicates a highway on which 600 vehicles per hour, mostly heavy trucks, can be accommodated at an assumed design speed of 30 miles per hour. Likewise, a 3000 P 50 classification indicates a highway capable of accommodating essentially 3,000 passenger cars per hour at a speed of 50 miles per hour.

The proper number, spacing and width of traffic lanes is an extremely important consideration in the segregation of motor vehicles and their safe use on the highways. The State highway systems of the United States comprise approximately 340,000 miles of highways that represent the main routes of highway travel in the United States. Of this total, 4,704 miles are three-lane highways, 3,082 miles are four-lane highways, and 221 miles are six-lane highways. Of the 3,303 miles of four- and six-lane highways only 604 miles are constructed so that traffic in opposing directions is separated by a raised parkway or median strip. The remainder or approximately 332,000 miles are of two-lane construction, and there is now no evidence that more than 95 percent of all roads in the State highway systems can or will need to be advanced beyond the stage of adequate two-lane improvement. A properly constructed two-lane highway is capable of carrying traffic of from 3,000 to 5,000 vehicles per day without serious congestion. There is, therefore, no need or economic justification to provide three- or four-lane construction where expected traffic densities are not likely to exceed those mentioned.

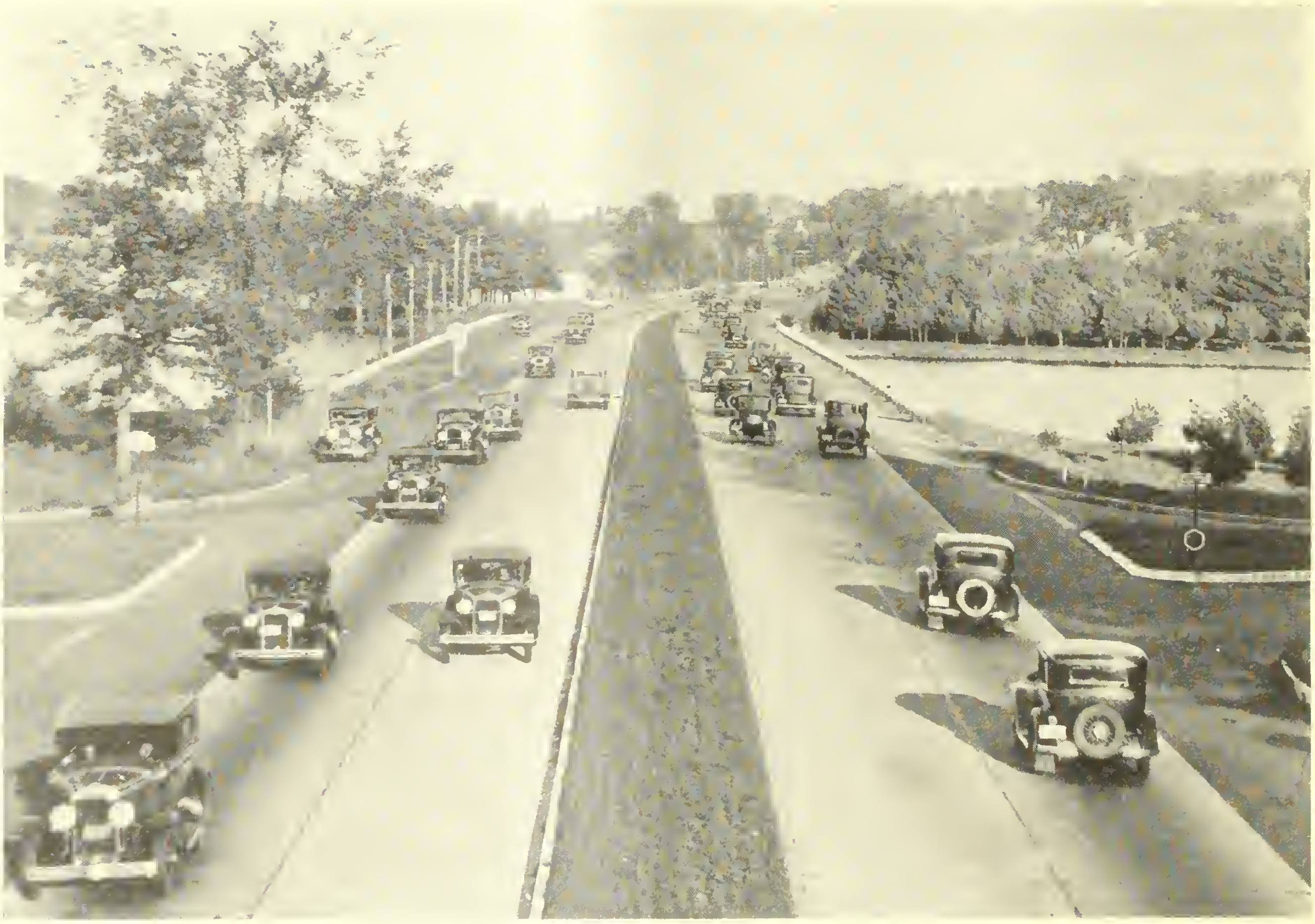
Opinion is sharply divided as to the wisdom of three-lane construction. In the vicinity of cities where the predominating flow of traffic is in one direction at different periods during the day and ample sight distance is available, three-lane highways add to the flexibility of movement and traffic capacity of the road. It is believed by some who have observed the operation of three-lane highways that as traffic increases the hazards increase at a greater ratio than in the case of the two-lane or four-lane highways. However, there appears to be no factual data indicating that the number of accidents on three-lane highways is greater than the number of accidents on two-lane highways when the number of vehicles on each is less than enough to fill the road and to make turning out impracticable.

In general, a three-lane highway for safe usage should have very good sight distances at frequent intervals to permit passing at high speed. Present knowledge and experience does not support a contention that a properly designed three-lane highway has no place in the highway program. While it may not be wholly desirable, economic considerations cannot be disregarded. Certainly a three-lane highway that lends itself to future expansion into a four-lane divided highway seems to be a logical phase of stage development. In the State of California three-lane highways are being constructed as part of a program of progressive stage development to afford increased capacity, better service to traffic and better economic use of the funds available. A greater mileage of improved facilities is being provided by stage construction than would be possible in using available funds for an ultimate divided four-lane highway.

A Three-Lane Dual Pavement on U S 30 in Pennsylvania.

Designs for these three-lane pavements in the State of California incorporate features which make them readily convertible into divided four-lane highways. Outside lanes are of portland-cement concrete and the middle lane is of less costly bituminous construction. The outer permanent lanes carry most of the traffic load and the center lane is used primarily for passing purposes. When future expansion to a four-lane highway becomes necessary because of increased traffic, the surfacing material of the center lane will be removed and salvaged for shoulder treatment. The original center lane then becomes the dividing strip for planting and the two permanent lanes become the inside lanes of a four-lane divided highway after two new outside lanes of concrete have been added. The center lane is usually of a darker color than the outer lanes and the contrasting color and difference in surface texture seem to prevent its indiscriminate use for other-than-passing purposes. Without adequate sight distance the three-lane highway is little better than a two-lane highway.

Three and four-lane highways of dual type surfacing using bituminous mixtures for interior lanes and portland cement concrete for exterior lanes have been used



A Four-Lane Divided Highway in Massachusetts.

for a number of years in the States of Massachusetts and Pennsylvania with considerable satisfaction.

When traffic is sufficient in volume to require the use of 4 traffic lanes the physical separation of traffic moving in opposite directions is desirable to reduce traffic hazards. A raised diving strip is preferable to a median strip flush with the elevation of the adjacent pavement. The width of the separating strip and its treatment are important. Raised separations may be from 4 to 6 feet in width, or from 12 to 30 feet or more in width. Sections 4 to 6 feet wide are too narrow for planting and should be paved, preferably with an all-weather type of surfacing of a color contrasting with the traffic lanes. This is to reduce maintenance of surface and emphasize its presence. The width, however, should not be so great as to permit the use of the separating strip for the passing or parking of vehicles. A separation 12 to 30 feet or more in width is ample to support a vegetative cover that will serve to prevent its use by vehicles except in emergencies.

The distance between roads need not be uniform. Advantage may be taken of the peculiarities of the topography and available right-of-way to construct two highways with sweeping graceful alinement and avoid the bare hardness in appearance resulting from uniform widths, grades, and side slopes. The danger and undesirability of a monotonous road also may be avoided by variable widths and differences in grade between roads.

The wider dividing strips are to be preferred to the narrow dividing strips for the reason that they reduce the traffic hazards caused by glare from headlights, particularly when planted with low-growing vegetation, such as shrubs or low-growing trees. They also provide at intersections an intermediate stop zone between the two roadways for crossing traffic and for turning traffic.

On two- and three-lane highways physical separation of opposing traffic is impracticable. On four, and six-lane undivided highways median strips to separate opposing traffic may be considered. However, new hazards should not be created when attempting to reduce existing hazards. In such cases the value of a vertical curb as a desirable and safe method of controlling traffic is open to serious question. Separators should be designed so as to permit vehicles traveling in the same direction to overtake and pass each other in safety without hazards which are likely to result from colliding with or sideswiping a curb and without encouraging vehicles to veer from it and thus reduce the effective width of surfacing. The high curb with vertical face commonly used in urban areas around dividing parkways is not satisfactory to use on rural, high-speed highways. It is preferable on rural highways to use a curb or other type of separator designed to warn drivers encroaching on the median neutral strip but to permit a vehicle to ride over it with a minimum of hazard in cases of emergency.

The question of what constitutes suitable or necessary road widths as determined by a study of the passing of vehicles on 2-lane highways was reported in the September 1937 issue of Public Roads published by the United States Bureau of Public Roads. The conclusions of that study indicate that pavements of 18-foot width are too narrow for modern passenger cars alone or for mixed traffic. Pavements of 20-foot width are reasonably adequate for light traffic roads used infrequently by wide trucks, but are inadequate for mixed traffic. Pavements of 22-foot width are entirely adequate for modern mixed traffic. It is also evident from the study that further increases in speed of vehicles will tend to make necessary road widths greater than 22 feet. A number of States already are using for two-lane roads a width of 24 feet or more.

The behavior of traffic on four- and six-lane highways depends largely upon whether the highway is urban or rural in character. On rural highways all traffic tends to use the outside lanes and to pass on the inside lanes. In urban areas, particularly where vehicles are permitted to park along the outer curbs, traffic tends to follow the inner or midlanes, usually, of course, to avoid slow moving traffic and hazards encountered near curbs.

The State of California is giving intensive study to proper laning of highways, particularly in the design of divided highways.

In some designs lanes 12 feet wide are used adjacent to the dividing strip, and lanes 11 feet wide are used adjacent to the roadway shoulder. The 12-foot width for the inside lane provides a greater operating space for vehicles overtaking and passing and thus reduces the possibility of sideswipes or other accidents. The outer 11-foot lane because of its proximity to the shoulder is entirely adequate for traffic in motion and the adjacent shoulders may be used for parking or emergency purposes.

In urban areas traffic tends to travel at appreciable distances from the edge of pavement particularly where curbs are used. Outside lanes, therefore, equal in width or wider than the inside lanes often are used to prevent encroachment on the inside lanes and to permit busses and other vehicles to approach and leave curbs without seriously interrupting the higher speed traffic.

Where extra width of outer lane or continuously wide shoulders cannot be had, provision should be made at predetermined intervals for widened areas in which busses may receive and discharge passengers or service trucks may load and unload merchandise without interrupting through traffic. To assist in segregating traffic and keeping vehicles in their proper lane, broad black, white, or bright yellow lines are painted along edges of lanes. The results are fairly good but the method is not effective against the irresponsible driver who uses the middle of the road or straddles the lane markers.

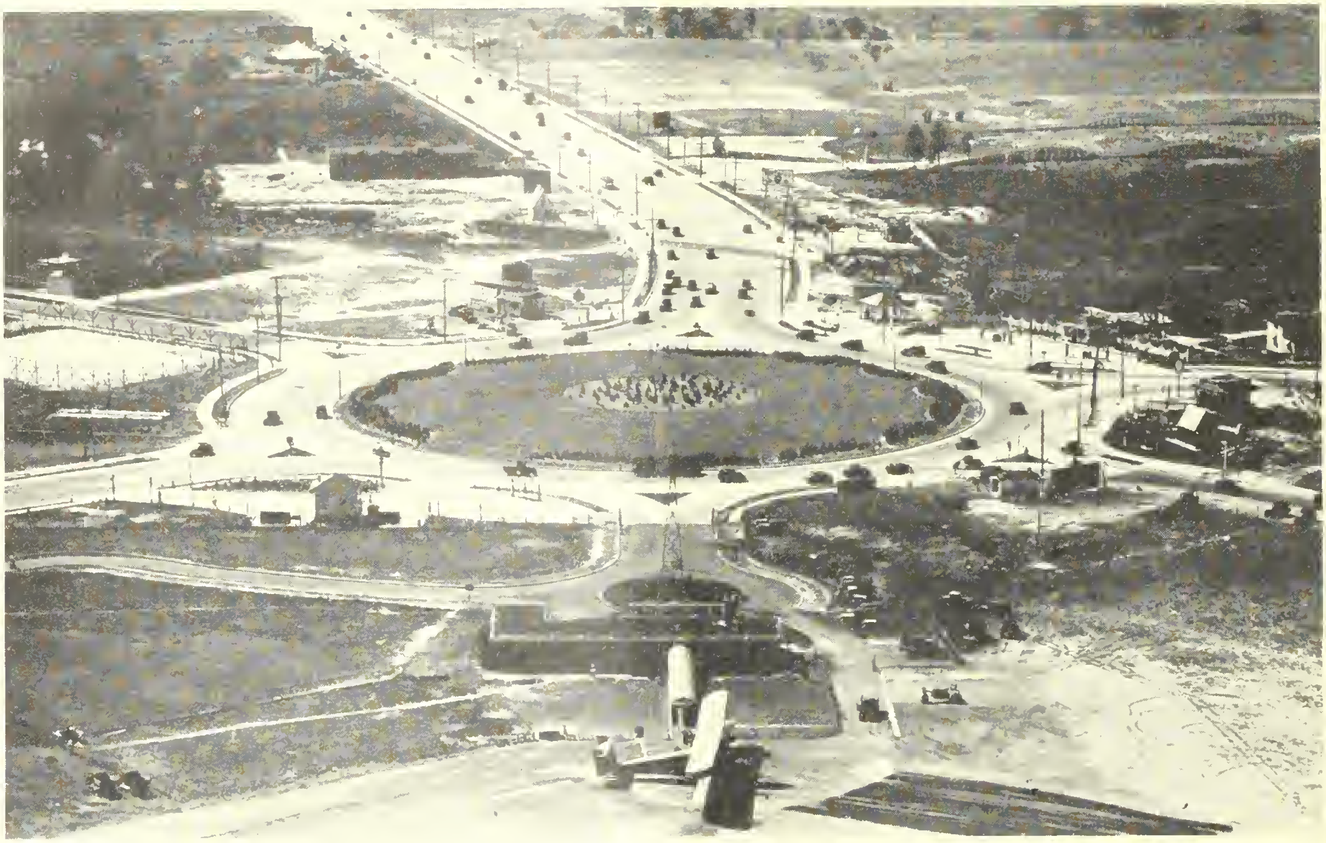
The State of New Jersey has undertaken many major improvements to reduce traffic accidents on its State highway system. Studies of accident records before and after the improvements were made are enlightening and in some cases contrary to common belief. An analysis of accidents occurring on roadways having hard surfaces of two-lane, of three-lane, and of four-lane widths showed as an average for all sections an accident rate of 3.10 accidents per million vehicle-miles traveled; on two-lane roadways 2.75; on three-lane roadways 3.53; and on roadways having four contiguous lanes the number of accidents was 3.61 per million vehicle-miles. In other words, an increased accident rate ocurred with increased width on roadways on which traffic lanes are contiguous. It is interesting to note that contrary to general belief the study showed that four-lane highways of the undivided type are more hazardous than three-lane highways. The apparent reason is that the wider road makes possible excessive weaving, that is, the uncontrolled movement of traffic from one lane to another.

Officials of the State of New Jersey obtained interesting data relating to accidents for sections of highways that had been converted from an undivided highway to a divided type. The following examples are cited: A study was made on a 2.1-mile section of State Highway Route No. 26 (US 1) in the township of New Brunswick where the change from a four-lane undivided highway to a four-lane divided highway was accomplished by the extremely novel method of using compressed air in fire hose to force apart and slide over one outer lane of the existing concrete pavement, thus providing space for a 12 foot center island. The addition of another lane outside the moved section made available a two-lane roadway on both sides of the island. After the widening, observations indicated a reduction of 36 percent in accidents of all types. With the exception of the construction of the dividing strip and wider roadway the characteristics of the converted highway remained the same as before.

Moving an Existing Concrete Slab to Provide a Four-Lane Divided Highway on US 1, 16 Miles North of Trenton, N. J.

Another example is a 7-mile section of State Highway Route No. 25 in Cinnaminson and Delran Townships where a two-lane road was converted into a four-lane divided road. On this section there was found a total reduction of more than 46 percent in all types of accidents. This case, however, is somewhat different from that of Route No. 26 for the reason that in addition to the construction of the 10- to 12-foot center island separating the original traffic lanes from the two new traffic lanes, alinement also was improved. It may be concluded from these and other reports that on multiple-lane highways segregating vehicular traffic in each direction is conducive to increased safety, convenience, and utility.

Bypass roads.—Where main highways pass through urban areas it is inevitable that they are crossed by numerous streets. Each crossing, with its conflicts between through, cross, and turning traffic, constitutes a hindrance to smooth operation and a hazard to life and property. The accessibility of a main highway



A Rotary Intersection in New Jersey.

encourages the commercial development of the roadside, the servicing of which adds to the congestion and hazard. It also is inevitable that the hazard to life and property, loss of time, inconvenience, and general confusion increase with the size of the city and the density of through and cross traffic.

A study of the problem shows that the original routings through many cities or towns were not objectionable when traffic was light and congestion unknown. In those early days when traffic was largely local in character, there were few improved roads and nearly all roads of importance passed directly through business centers of communities. When engineers first proposed traffic relief through bypass roads around towns and cities, merchants along the main streets strongly opposed such proposals claiming that valuable transient business would be lost thereby. To meet the needs of through traffic which neither belonged on the local streets nor desired to use them, and at the same time meet the needs of local traffic and satisfy the local businessmen, roads have been improved through the communities and bypass roads later constructed skirting the built-up sections so that the traveler may select the route to follow.

The belt-line or bypass road has proved an effective means of segregating through traffic from local traffic, expediting traffic, and reducing accidents. However, unless access from abutting properties on bypass roads is controlled by the State, the continued development of such frontage for business and residential purposes gradually tends to reproduce the very traffic conditions that the bypass road was designed to alleviate.

Widening of a highway instead of constructing a bypass road will not always satisfy the demands of traffic of mixed character. Moreover, widening and straightening local streets to accommodate through traffic is costly, although relatively minor improvements in many cases will make them entirely adequate to handle local and short-distance traffic. Safe, high-speed operation cannot be obtained where vehicles are parked along curbs or in operating lanes, nor can it be expected in built-up communities where adjacent property owners have unrestricted right of access.

In St. Louis, Mo., the major thoroughfare system was developed by widening existing streets to as much as 76 feet, thus providing six traffic lanes. It was found that traffic did not uniformly distribute itself over the six-lane main highway. The largest volume traveled near the center of the roadway, a smaller volume traveled in the second lanes adjacent to parked cars. The principal improvement apparent from the widening was less congestion at intersections where the extra space was occupied by cars awaiting the go signal.

Bypassing centers of population is an established policy of the highway department of the State of New Jersey and is generally satisfactory to through and local traffic. It has been found that bypass routes around New Jersey cities shorten distances for traffic destined beyond the municipality bypassed, improve the alinement of the main route, separate through and local traffic, eliminate the costly operation of widening existing streets, provide double traffic routes where needed, and reduce danger to local traffic and pedestrians. Although numerous examples of bypassing cities in that State can be cited, only the Keyport bypass is mentioned here. One of the most annoying and dangerous obstacles to travel by way of the seashore route from the New York City, Jersey City, and Newark area to the coastal resorts to the south, was removed by the Keyport bypass. The old route through the borough passed over narrow streets and included a railroad grade crossing carrying many trains. Before construction of the bypass road traffic up to 47,529 vehicles daily wormed its way through the city and on busy days a delay of 2 hours was not unusual. The bypass eliminated all traffic hazards and delays and justified the cost of $1,000,000 for the 2.6 mile improvement.

Where bypasses of the limited or freeway type are in use they have demonstrated an ability to carry large volumes of traffic around highly congested areas without appreciable reduction in speed and under conditions approximating complete safety.

Traffic on any heavily traveled highway in which there is no segregation with respect to vehicle types frequently comprises heavy freight trucks, light freight trucks, light service trucks, passenger automobiles, and pedestrians. Each class of traffic should be able to function efficiently, conveniently, and safely. If the volume of traffic on a highway is great enough, slow-moving traffic may be segregated from fast-moving traffic by the construction of separate roads. The most practical means of segregating slow-moving and fast-moving traffic is by restricting roads for fast-moving traffic to passenger vehicles, though there appears to be an increasing tendency to construct and maintain high-speed trucks particularly of the lighter weights.

Frequently bypass roads are restricted to passenger vehicles, thus segregating high-speed from slow-speed traffic and segregating passenger-vehicle through traffic from passenger-vehicle local traffic, but complete vehicular segregation is not accomplished because all truck traffic is required to use the road nominally devoted to local traffic. The Pulaski Skyway, an elevated bypass route in the State of New Jersey approaching New York City, was first opened to mixed traffic. In spite of no grade crossings, side entrances, or other hindrances, smooth operation did not result. Trucks now are prohibited and daily traffic densities of more than 50,000 vehicles on this four-lane highway frequently are recorded.

Freeways and parkways.—A freeway is a roadway in which the traveled surface is devoted to the exclusive use of motor vehicles; direct access to and from abutting properties is eliminated; access is restricted to properly located and designed entrances and exits at infrequent intervals; all intersections are separated or eliminated so that no traffic crosses at grade; and opposing streams of traffic are separated physically.

A parkway is a freeway with recreational facilities, parks, and scenic areas provided by the acquisition and development of an appreciable though variable width of right-of-way.

In this country an insignificant portion only of the improved highway mileage has been developed to the point of separating all traffic at intersections by means of overpasses or underpasses and only a relatively small mileage will need such improvement. To improve 5 percent of the State highway systems, comprising 340,000 miles, with four or more traffic lanes, with opposing traffic separated, grades at intersecting highways separated, border roads provided for unrestricted access from abutting property and sidewalks constructed where needed, would involve an expenditure of approximately $4,000,000,000. Except for relatively short sections, principally in the vicinity of New York City, and the Merritt Parkway in the nearby State of Connecticut, no freeways of appreciable length have been constructed in this country. However, the needs of traffic are forcing attention to this type of development as is evidenced by efforts on the part of some of the States to seek legislation to make possible this type of highway facility.

Cycle tracks, bridle paths, and provisions for horse-drawn traffic.—After some 30 years of relatively little use, the bicycle is coming back into favor in the United States. Its use is growing to such an extent that communities may well consider provision for this convenient and inexpensive vehicle. The estimated total number of bicycles in use in 1936 was 10,000,000. In that year the number of bicycles manufactured was 600,000 more than in 1933. At present, however, bicycles are used almost exclusively for recreational purposes and rarely as means of transportation except for very short distances such as to and from school.

With the increase in bicycle use has come a toll of accidents. In 1935 there were 13,000 accidents involving motor vehicles and bicycles resulting in 350 fatalities. From 85 to 95 percent of all the fatal accidents of this character occurs in urban areas. The most common types of collisions are those in which motorists run down insufficiently lighted bicycles at night, and those caused by cyclists suddenly entering the roadway from a sidewalk or driveway.

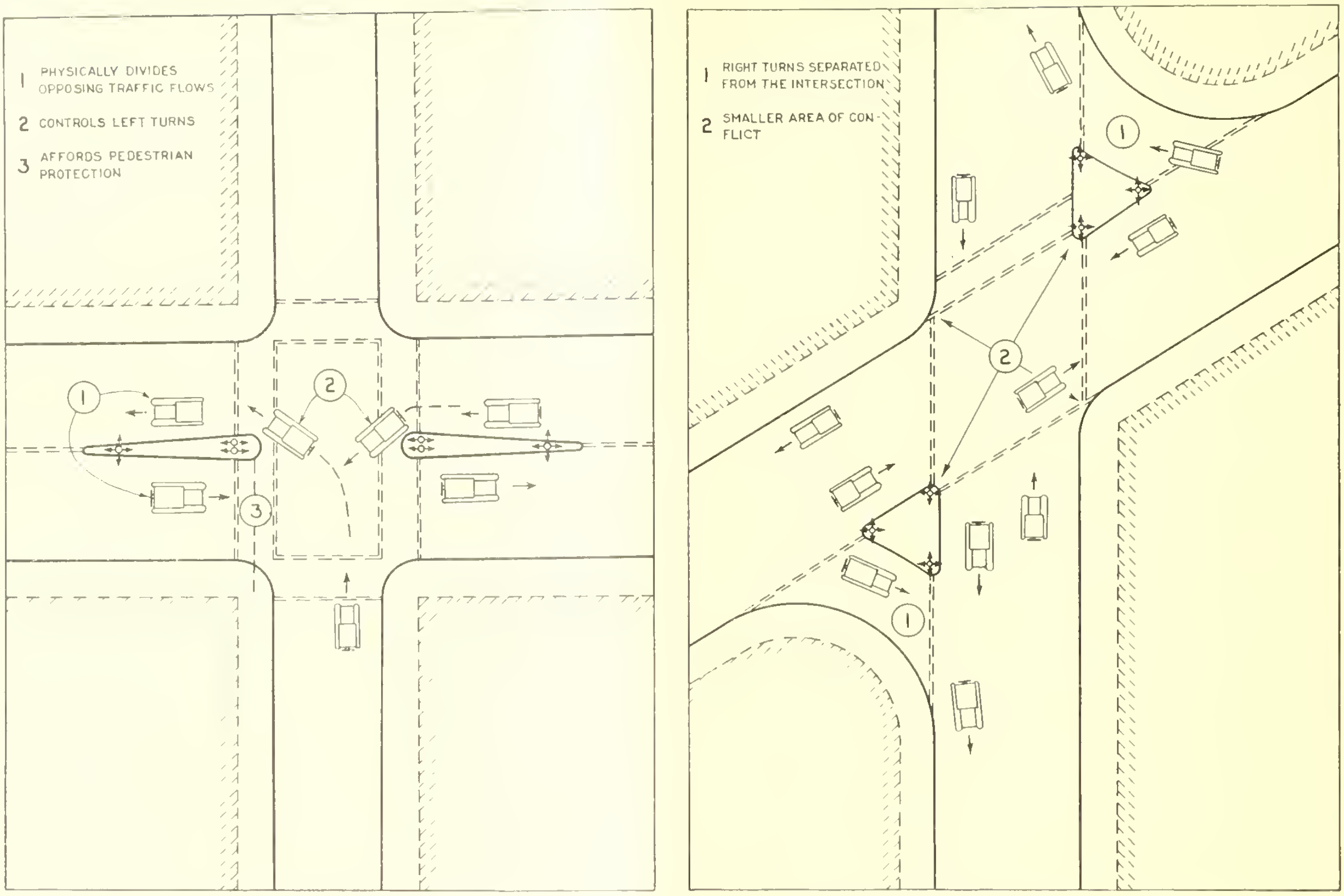
There have been few, if any, tracks constructed in the United States intended for the exclusive use of bicycles. As with footways, bicycle tracks should be entirely separated from the roadway. If there are few bicycles and few pedestrians, both may use the same pathway. In general, however, each should have its own track. The bicycle would probably be more extensively used in this country were it not for the extreme danger of cycling on streets and highways and because of legislation prohibiting its use on sidewalks.

The construction of bridle paths in this country has been confined almost exclusively to park areas in or adjacent to the larger centers of population. They should be kept a considerable distance from main traveled highways. There is no good reason why they should parallel highways except for the purpose of using the same right-of-way, as they do not have any close connection with an automobile highway. Where bridle paths cross main highways in park areas it frequently is common practice to protect such crossings by traffic signals that can be operated by equestrians approaching the crossing.

Horse-drawn traffic has practically disappeared from the main, trunk-line highways in the United States in spite of the fact that the horse is still an important factor in agricultural production. Modern highway structures are designed almost solely for motor traffic and are not well adapted to horses. In a relatively few States earth shoulders of extra width have been constructed to accommodate horse-drawn vehicles but these cases are rare and confined largely to relatively flat country where cost of grading is low.

Horse-drawn vehicles, flocks of sheep, and herds of cattle still may legally use the highways and sometimes do, but this type of traffic is confined almost exclusively to the farm and its adjacent earth roads.

Footways and sidewalks.—Sidewalks are common to city streets but walkways in rural areas have been constructed only to a limited extent, requiring pedestrians


1 PHYSICALLY DIVIDES OPPOSING TRAFFIC FLOWS
2 CONTROLS LEFT TURNS
3 AFFORDS PEDESTRIAN PROTECTION
1
2
3
1 RIGHT TURNS SEPARATED FROM THE INTERSECTION
2 SMALLER AREA OF CONFLICT
1
2
1

CHANNELIZING RAISED TRAFFIC ISLANDS

BYPASS AND CHANNELIZING RAISED TRAFFIC ISLANDS

Channelizing Islands as Used on City Streets in Milwaukee, Wis.

to use the vehicular roadway. With the increase in the speed of travel and the density of vehicular traffic the vehicle-pedestrian hazard has increased and there is a growing movement for segregating pedestrian from vehicular traffic in rural areas by the construction of walkways.

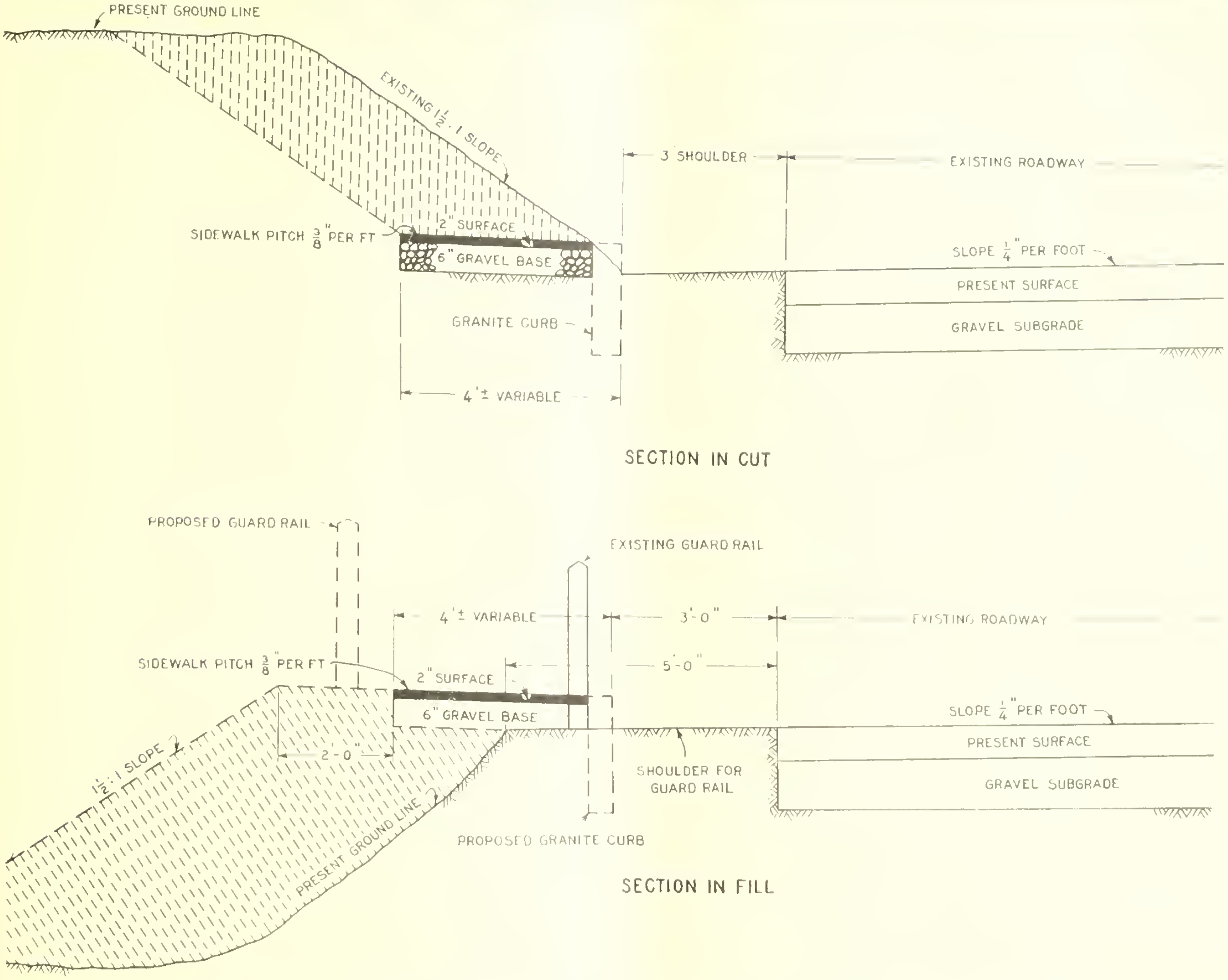
The "right-of-way" or strip of land dedicated to the use of the public for travel, is generally wide enough to allow for building sidewalks well outside the paved strip which is built for vehicles. On every important highway there should be such a pathway and if the needs of the walking public in the community demand it, sidewalks should be built on both sides of the street. They should, if possible, be several feet away from the vehicle pavement and the separation emphasized by a line of intervening shrubbery or trees. This serves the purpose of making it practically impossible for the vehicle driver to encroach on the space reserved for pedestrians and at the same time discourages pedestrians from wandering on or too near to the motorway.

Sidewalks should be so located, designed, and constructed that pedestrians will use them. They should attract pedestrian traffic from the highway to themselves. Unless the sidewalk is attractive and convenient the pedestrian will not use it. The use of two relatively narrow sidewalks on rural highways where pedestrians are encouraged to walk facing the traffic, may, in some cases, be more desirable than one sidewalk which compels pedestrians, frequently children, to cross a busy highway. Pedestrians should be able safely and conveniently to walk along the highway in either direction and to cross the highway at reasonably frequent intervals.

Sidewalks must be easily accessible to the roadway pavement without the necessity of wading through snow, slush, mud, or water. The question of snow removal from sidewalks as well as pavement, therefore, should be considered in snow States. In flat and rolling country where grading is relatively light, sidewalks may be located farther from the pavement than where grading is heavy and more costly, such as on relatively narrow fills with guardrails, through swamp or marsh land or in cuts and fills in mountainous areas and on bridges.

Vehicle-pedestrian accidents in urban areas generally occur while pedestrians cross the street. There is an increasing tendency to prohibit crossing of streets except at intersections where the crossings for pedestrians are marked clearly. Where pedestrian traffic density warrants it, traffic lights are adjusted to partially subordinate vehicular traffic to pedestrian traffic by inserting a time interval for pedestrians or restricting vehicular turning movements at a limited number of crossings.

Sometimes islands to separate opposing traffic at crossings are constructed at intersections of wide city streets so that pedestrians crossing the street may use them for refuge and need watch traffic in only one direction at a time. They also confine left-turning vehicular traffic to a limited width of street. Islands generally are unsatisfactory at intersections on rural highways because of (1) insufficient roadway width,



Cross Sections of Typical Sidewalk Designs in Cut and in Fill Used in Massachusetts.

unless adequate provision is made for gradual widening approaching the intersection, and (2) the additional hazard to high-speed vehicular traffic.

Places where the highway may be crossed by pedestrians should be clearly marked. These guiding marks should be made as clear and definite for the motorist who is approaching the crossing as for the pedestrian. Traffic signs for this purpose may be set on posts at the side of the pavement or painted on the pavement itself. They should be unmistakably clear—a single short word or phrase or, better still, a symbol whose meaning can be grasped at first glance.

Sidewalks or footways should be planned as an integral part of the roadway wherever their need is indicated. There are no available statistics indicating what traffic and what speeds justify the construction of one sidewalk or two sidewalks or of any sidewalk at all. In general, however, sidewalks are needed within and adjacent to industrial areas, along roads connecting villages situated relatively close to each other, in the vicinity of large schools, summer resort areas, unincorporated suburban areas, and elsewhere where considerable pedestrian traffic exists or may be expected.

The cost of grade separations for pedestrians usually cannot be justified, and experience has shown an unwillingness on their part to use either overhead bridges or underpasses. For this reason, safety islands or refuges for pedestrians crossing wide highways or wide streets in urban areas are likely to serve the greatest number.

Statistics available for the State of Pennsylvania show that 44 percent of the people killed by motor vehicles in that State in 1936 were pedestrians. A large percentage of the pedestrian fatalities were in built-up sections where sidewalks existed, indicating a need for a careful study of possible traffic control for pedestrian crossings.

The Department of Public Works of the State of Massachusetts has completed a State-wide program of construction calling for 500 miles of sidewalks along the State highway system of 1,890 miles for which the State appropriated $4,000,000. The purposes of this undertaking were to reduce accidents to pedestrians, provide employment, and indirectly to increase the value of abutting property. After the program was under way property owners offered easements for sidewalks without compensation.

The locations for sidewalk construction were based on a 6-year study of collisions between pedestrians and motor vehicles in the State from 1930 through 1935. For that period the study showed a total of 4,427 accidents of the above class in which 3,841 persons were injured and 586 killed. Before the program of construction began in 1935 there was a progressive increase in the accident rate as well as a general increase in pedestrian accidents in rural counties as against a decrease in the larger cities.

A table of pedestrian accidents on routes where sidewalks were constructed gives a comparison of pedestrian injuries and deaths for the first 6 months of each year since 1933 (table 1).

TABLE 1.—*Pedestrian injuries and deaths on routes where sidewalks were constructed*

First 6 months of—	Injuries	Deaths
1934	240	21
1935	243	26
1936	212	24
1937	154	18

Records for the first 6 months of 1935, which was the period before the sidewalk program started, and the first 6 months of 1937, when the major part of the work had been completed, clearly indicate a decrease in pedestrian casualties.

Despite an increase during this period of 12 percent in registrations and 15 percent in gasoline consumption, the number of pedestrians injured on these State highway routes decreased over 36 percent, whereas total pedestrian accidents on all Massachusetts roads for the same period decreased only 11 percent.

Reductions in pedestrian fatalities on State highways are consistent with those in injury cases. On State highway routes where sidewalks were constructed, the pedestrian fatalities in the same 6-month periods showed a reduction from 26 to 18, or about 31 percent. This decrease is most significant when one considers that in the same period pedestrian fatalities in the State as a whole increased from 188 to 201, or about 6.9 percent.

The above figures are for all types of pedestrian accidents. Studies of accidents involving pedestrians walking along the highway show a general reduction of over 50 percent, and on many routes where sidewalks have been built, no accidents of this type have been recorded during the first 6 months of 1937, where formerly there were many serious casualties annually.

All new projects for the construction or reconstruction of State highways in Massachusetts now include the construction of sidewalks as an integral part of the design, except in unsettled areas where pedestrian movements are negligible. Even in these cases, provision is made in the design for future construction of sidewalks to allow for possible future development of abutting property. The present plan is to construct at least one continuous sidewalk for the entire length of each highway, except in undeveloped country, whereas the former plan was to construct no sidewalks at all except in highly developed country. The State authorities feel that the saving in life as shown by the above studies justifies a considerable expenditure of State funds for walkways.

Service roads and parking places.—In metropolitan areas additional facility and protection must be provided on main routes to reduce hazards, prevent congestion, and insure a free and easy flow of traffic. This need has come about largely because of the dispersion of city population in suburban areas.

Experience has shown that widening a roadway by adding more traffic lanes adjacent to those already in place does not increase roadway capacity in proportion to the traffic lanes added and does not reduce accidents to the extent desired. The capacity per lane decreases with an increase in the number of contiguous lanes beyond four, dropping sharply beyond six. In urban areas vehicles parking at and leaving curbs as well as those occasionally double-parked or cruising in front of business establishments slow down both through and local traffic and constitute serious hazards.

The bypass road often may not fully accomplish the object of diverting a considerable percentage of traffic from an existing through route. The destination of much of the traffic may be within the city and local conditions, such as extended areas of urban development, may make it difficult to construct a bypass road attractive to through traffic destined beyond the city. Under such conditions the existing routes through cities must be improved.

Through routes have been successfully improved and local traffic segregated from through traffic by providing roadway surfaces adjacent to abutting properties and separated from through traffic lanes in the central portion of the street by appropriate dividing strips. The roadways adjacent to abutting property may be termed service roads and in combination with the central, through-traffic lanes form the typical boulevard sections of many large cities.

In most urban areas existing streets generally are inadequate for the parking of vehicles. This is particularly true in commercial sections where parking facilities often are necessary for the success of the business establishments. Limiting parking on streets, parking meters, and other forms of enforcement relieve this situation to a limited extent. Garages expressly for parking have been constructed and much valuable land in the centers of many cities has been converted into pay parking lots. Many large business establishments have recognized the need for parking areas and have found it profitable to construct such areas on adjacent private property. Many others have found it necessary to move their establishments or establish branch offices in suburban areas where parking is provided readily. Small retail establishments have found it profitable to pool their resources to provide parking areas known as "park and shop" areas which serve a number of establishments at one time. On high-speed through routes in rural areas establishments such as eating places have found it inexpensive and desirable to provide parking areas for their patrons off the highway.

The need for providing parking areas off the public highways is being recognized to the extent that some municipalities have adopted zoning ordinances which require certain types of structures, such as theaters and apartment houses, to provide garages or parking areas in proportion to the number of persons for which the structures are designed.

Road junctions and crossings.—When two or more roads cross each other traffic hazards and impediments to smooth operation are created by the conflict between through and turning traffic on both roads. The ideal treatment of an intersection is one which results in the segregation of different types of traffic and the segregation of traffic in each and every direction. This is not always practical nor is it often economically advisable. The treatment of each intersection depends principally upon the density of traffic on each road and the density of traffic making each possible turn, but considerable leeway must be permitted the designer because of the effects of topography, cost, and difficulty of procuring adequate right-of-way, probable speed of traffic, and other factors. Individual judgment is necessary also because thought is not as yet crystallized on the traffic densities for through and the various turning movements which justify the construction of the various types of intersections. A few fundamental principles, however, have been fairly well established.

When two highways intersect at grade, consideration of the traffic volume on each highway is the most important factor governing the design of the intersection. There are two general conditions, one in which the traffic volume is approximately equal on both roads and the other in which the traffic volume is greater on one highway than on the other. In the first case, unless stop signs are used, a driver on either highway should be able to see conditions at the intersection in sufficient time to allow the vehicle, if necessary, to be brought to a stop before reaching the intersection. Where visibility and sight distance are inadequate to accomplish this, one road should be subordinated to the other road by stop signs or slow signs, or stop signs used on both highways. In the second case movement on the road carrying the greater traffic volume should be given preference over the road carrying the lesser volume by the use of stop signs on the less important road.

The proper use of signs at intersections is most important and their selection should be based on sound engineering principles established by actual studies of accidents, speeds, delays, sight distance, braking distance, and physical conditions. Unfortunately there is a wide variation in official responsibility for the selection, installation, and maintenance of traffic control devices, particularly in metropolitan areas. Slow signs should indicate the safe approach speed and be placed far enough in advance of an intersection to enable a vehicle traveling at the speed for which the highway is designed to slow down to the speed indicated on the sign before reaching the critical point of the intersection. In no case should sight distance be less than the distance in which the vehicle can be stopped. The ability to see an appreciable length of the intersecting highway is fully as important as the ability to see on horizontal and vertical curves along a highway between intersections.

Experience in the State of Massachusetts indicates that to avoid collisions at intersections stop signs should be used where sight distance is so short as to require a 10-mile-per-hour speed.

Intersection designs preferably should enable traffic to enter slowly and to leave quickly. Approach visibility to an intersection having a high percentage of turning movement is of prime importance. The elements of uncertainty and surprise should be eliminated. An intersection should be so marked that no doubt can exist in the driver's mind as to what is the right thing to do. Intersections which readily can be recognized by a motorist as requiring caution have a more favorable accident record than those in which conditions are not clear. The lay-out of an intersection should favor the larger volumes of traffic movement. The roads approaching intersections at grade should be flared or widened but not to the extent of causing traffic to wander from its proper channel and cause congestion.

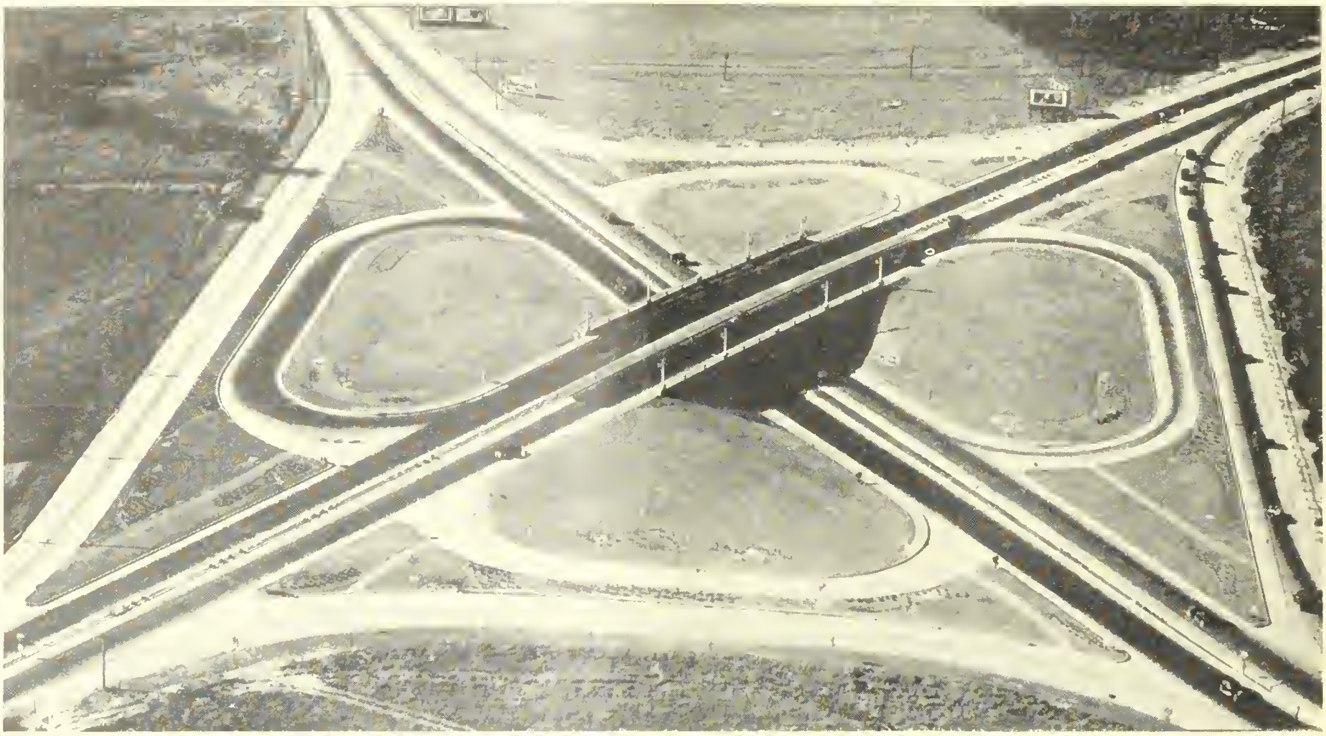
Heavy volumes of traffic may require installation of traffic lights and a greater widening of intersection than is needed for lesser traffic volumes. There should in any case be sufficient space in which to accomplish traffic movements without the necessity of encroaching on adjacent traffic lanes. Traffic lanes at intersections should be so located that it will be easy for traffic to make natural and smooth progress free from any abrupt changes in direction that are likely to cause collisions or sideswiping. Automatic control devices of a number of types have been used with excellent results, and the lighting at night of busy intersections has evident advantages. The character of control devices in use includes warning flashers of several types including the pretimed, the vehicle-pedestrian actuated, synchronous-progressive, and others. The selection of type of control device is dependent on local conditions, and the locations where signals are contemplated should be given careful field study before the adoption of one type or another. The vehicle actuated type of control is particularly effective in facilitating infrequent cross movements at intersections where the predominant traffic flow is on one route.

Rotary traffic may be defined as the movement of vehicles in one direction around a central area. The central area together with the one-way road around it and with the entrances and exits generally is called a "traffic circle" though it may more accurately be termed a traffic loop or circuit. It is usually used at the convergence of several streets or highways.

The circle is in effect a continuous series of one-way roads connecting the entrances and exits of the various intersecting roads. The length between roads on the one-way road or traffic circle should be about 250 feet and should be not less than 150 feet in order that traffic may weave from the entrance to the inside lane and back again as required at the moderate speeds prevailing on traffic circles. They may be longer but preferably not more than 700 or 800 feet.

The variation in the angles and locations of the several converging roads and the necessity for distorting the traffic circle to provide the minimum distances between roads, to avoid expensive parcels of land, and to avoid expensive construction results in so-called "traffic circles" of various shapes such as true circles, rounded squares, rectangles, triangles, or other polygons, ellipses, and many other unsymmetrical shapes. Parts of existing streets or roads may be included. Each road entering a "traffic circle" in rural areas should be divided into an odd-shaped Y by a conspicuous, distorted triangular island which performs the important function of guiding traffic to the right and making all one-way roads funnel-shaped, that is, wide at the entrance and narrow at the exit. The islands also may be used for signs and as stopping places for pedestrian traffic. The curbs generally are constructed with slight slopes so that the islands may be mounted in emergencies.

The four-lane highway with traffic in opposing directions separated by a parkway may be designed to decrease hazards at intersections at grade. On heavily traveled four-lane roads having adjacent lanes, it is difficult to obtain a sufficient break in traffic to permit safe crossing unless traffic control lights are used. If the roadways are separated a sufficient width to provide a safety island between the opposing lanes of traffic, cross movement may be effected in two operations. A break in the flow of traffic in one direction permits a vehicle to reach a point in the pavement adjacent to the safety island where a stop may be made and the crossing completed when there is a break in traffic in the opposite direction. The width of separation between opposing lanes of traffic should be not less than 30 feet for this



A Cloverleaf Intersection in New Jersey.

purpose. Such construction also permits the openings between islands to be used for turn-around purposes. Divided highways of this type offer a much safer crossing than an undivided highway and also offer a partial solution to the problem of handling traffic making left turns from main roads into secondary roads.

Accident statistics for the State of New Jersey show that rotary circles facilitate traffic flow better than intersections controlled by traffic lights and with fewer accidents. A traffic circle can be designed to carry any reasonable volume of traffic, whereas a traffic-light controlled intersection has a capacity equal only to about 50 percent of the combined capacities of intersecting roads. Illumination of circles at night is necessary. Warning signs should be placed well in advance of traffic circles and large lighted signs are needed at circles to direct traffic. As a further aid to traffic direction, reflectorized units set into the curbs have been used to some extent.

The extent to which traffic circles are justified by traffic volume at any given intersection is a matter of judgment. There are no conclusive data on which there is general agreement, nor is information available on which to base a change from a traffic circle to a grade separation. Much depends upon the traffic, topography, cost of land, and construction. If sufficient right-of-way has been purchased for the ultimate construction of a grade separation with ramps, there is generally sufficient area for a traffic circle and the initial improvement may begin with a circle and later as necessity arises be developed into a grade separation with connecting ramps.

The separation of grades is a most effective method of increasing the traffic capacity of intersections, and of eliminating conflicting traffic movements and reducing accidents. A properly designed highway grade separation has approximately the same capacity as that of the roadways bringing traffic to it. Connecting roadways or ramps complicate the construction and facility of grade separations and should be used only where access and egress are definitely needed. Rates of grade and of curvature on ramps should make possible an easy and continuous flow of traffic without the close grouping of vehicles which creates hazards at points of entry to the main highways.

The need for ramps sometimes has been overestimated. Four ramps are not always needed and construction may begin with two ramps, when the volume of traffic on the less important road permits left turns without any particular hazard. In case future traffic requires four ramps, two more can be added later. Designs and right-of-ways purchased for original construction should, of course, make proper provision for later development. Ramps at grade separations sometimes are confusing. Signs visible day and night must be placed well in advance of a grade separation. They must inform drivers desiring to turn how to do so. The wording or diagram must be as simple as possible.

The highway department of the State of New Jersey recently made before-and-after records to determine the efficiency of highway intersections constructed in that State. In a comparison between a grade separation with ramps and a signalized intersection a short way distant on the same highway, it was found that despite a considerably greater volume of traffic on the grade separation there were fewer accidents there than at the signalized crossing. A similar comparison between a traffic circle and a signalized intersection was made which indicated that despite the constant operation of traffic-control signals at the signalized intersection there were 13 times as many accidents as in the same period at the traffic circle.

Highway grade separations in urban areas have been constructed for many years, two notable examples being the Grand Concourse in New York City on which the grades of all important cross streets were separated about 1916 and the first of the Westchester County parkways on which all crossings at grade were eliminated, the first bridges for this purpose being designed and constructed about 1914.

The first rural grade separation with ramps in the State of New Jersey was built in 1928. Its capacity was observed in 1930 when a 16-hour traffic count recorded a total of 62,527 vehicles passing through the intersection. The maximum hourly count was 6,074 vehicles or at the rate of more than 100 vehicles per minute. Eleven percent of the total traffic was right-turn traffic and 12 percent left-turn traffic. The State of New Jersey has 125 highway grade separations located for the most part on a few heavy-traffic routes. The highway that forms the main approach to the Holland tunnel under the Hudson River between Jersey City and New York City has a roadway without grade crossings for 9 miles that passes under and over many local roads. For part of its length this roadway is depressed in a rock cut and a part of the remainder is elevated on embankments or viaducts.

Summary.—

1. Two-lane highways properly designed and paved to a width of 22 feet are capable of carrying modern mixed traffic of from 3,000 to 5,000 vehicles per day.

2. Three-lane highways of the dual type having long sight distances for passing at frequent intervals are capable of carrying approximately two times as much traffic as two-lane highways.

3. Four-lane highways having connected lanes carrying two lanes of opposing traffic in each direction are unduly hazardous and their further construction for use by high-speed mixed traffic in rural areas is to be discouraged.

4. Four-lane highways with opposing traffic separated by a raised median or parking strip, preferably not less than 30 feet in width, practically eliminate head-on collisions, and reduce hazards at intersections at grade. Such construction is recommended.

5. A need for limited roadways of the freeway type is evidenced by traffic demands and legislative proposals for construction of this character.

6. Bypass roads constructed around built-up sections segregate through traffic from local traffic, remove it from city streets, and facilitate its free movement by eliminating hazards and reducing delays; but to be effective over long periods control over abutting property must be secured to prevent unrestricted access.

7. Inefficient intersections greatly reduce the capacity of highways and cause accidents. A limited widening of roadways at intersections with marked lanes and speed control signs with or without traffic control signals are reasonably effective for light to medium traffic volume. For heavier traffic volume traffic circles and grade separations with or without ramps are recommended.

8. In built-up sections traversed by through and local traffic some measure of relief from congestion is possible by the construction of service roads for the slow local traffic separated from the through traffic by raised strips or islands. Public parking lots and garages adjacent to main highways have been found to reduce hazards resulting from parking.

9. Studies of accidents involving pedestrians along the highways in the State of Massachusetts clearly indicate a substantial decrease in pedestrian casualties since the completion of 500 miles of sidewalk. Accidents involving pedestrians walking along the highway generally have been reduced 50 percent. On many roads formerly the scene of serious casualties no accidents since the construction of sidewalks were recorded for the first 6 months of the year 1937.

10. In the United States there is no immediate problem relating to the segregation of traffic using cycle tracks or bridle paths. Horse-drawn traffic is confined principally to light-traffic earth roads in agricultural areas.

STATE MOTOR-VEHICLE REGISTRATIONS, 1937

(Compiled for calendar year from reports of State authorities [1])

State	Registered motor vehicles, private and commercial [2]					Other registered vehicles		Publicly owned vehicles						Dealers' registrations and plates [7]		1936 total registered motor vehicles [8]	Year's change in motor-vehicle registrations	
		Passenger motor vehicles						Federal [5]			State, county, and municipal [6]							
	Total motor vehicles	Total	Automobiles (including taxicabs)	Motor busses [3]	Motor-trucks, tractor trucks, etc	Trailers and semi-trailers [4]	Motor-cycles	Motor vehicles	Trailers and semi-trailers	Motor-cycles	Motor vehicles	Trailers and semi-trailers	Motor-cycles	Regular registrations	Extra sets of plates		Increase or decrease	Percentage change
Alabama	313,359	257,218	253,653	3,595	56,111	4,850	830	1,917	44	14	3,856	----	191	2,914	----	297,292	16,067	5.4
Arizona	129,210	106,237	105,793	444	22,973	4,321	423	2,236	80	7	1,891	114	27	2,179	----	115,035	14,175	12.3
Arkansas	229,867	173,923	173,575	348	55,944	12,278	507	2,096	25	1	2,806	12	26	559	----	217,227	12,640	5.8
California	2,481,653	2,189,378	2,189,378	(8)	295,275	132,932	11,260	7,140	280	90	23,359	1,523	1,074	5,145	----	2,327,984	156,669	6.7
Colorado	337,217	282,123	281,080	1,043	55,094	1,468	1,316	1,899	12	22	(10)	(10)	(10)	3,492	----	316,050	21,167	6.7
Connecticut	436,564	368,473	367,413	1,060	68,091	4,904	1,996	669	5	1	3,672	67	230	2,844	7,831	(11) 403,263	33,301	8.3
Delaware	63,599	53,285	53,285	(9)	10,314	2,645	236	382	15	9	950	25	62	756	----	59,629	3,970	6.7
Florida	418,145	347,837	346,230	1,607	70,308	16,238	1,244	1,646	43	15	5,131	----	180	2,765	----	386,907	31,238	8.1
Georgia	441,847	363,641	363,233	408	78,206	12,262	1,085	3,255	81	49	(10)	(10)	(10)	3,069	----	410,583	31,264	7.6
Idaho	142,110	113,605	113,477	128	28,505	18,645	564	1,344	74	----	1,447	112	11	186	----	133,037	9,073	6.8
Illinois	1,768,946	1,554,567	1,554,567	(9)	214,379	23,210	5,827	3,484	137	25	8,395	265	663	4,329	----	1,659,750	109,196	6.6
Indiana	956,016	815,724	814,564	1,160	140,292	61,102	4,092	1,631	118	7	[12] 5,704	----	[12] 147	2,644	----	905,088	50,928	5.6
Iowa	745,602	657,734	657,734	----	87,868	[13] 66,172	2,292	1,225	17	3	[12] 5,393	[12] 300	[12] 75	2,069	----	728,414	17,188	2.4
Kansas	586,685	493,639	493,293	346	93,046	5,862	986	1,511	58	12	(10)	(10)	(10)	1,867	1,544	577,906	8,779	1.5
Kentucky	404,455	345,114	344,542	572	59,341	(14)	951	1,754	12	130	[12] 3,990	----	----	854	----	372,576	31,879	8.6
Louisiana	323,498	245,665	245,180	485	77,833	14,321	834	1,825	49	25	[12] 4,822	[12] 291	[12] 60	369	----	302,420	21,078	7.0
Maine	200,907	157,736	157,596	140	43,171	[15] 9,751	874	673	15	10	2,096	150	30	794	----	191,554	9,353	4.9
Maryland	387,410	332,928	331,944	984	54,482	3,422	1,549	2,082	95	57	(10)	(10)	(10)	9,375	----	378,462	8,948	2.4
Massachusetts	846,556	742,521	737,604	4,917	104,035	11,311	1,169	2,472	60	12	[16] 5,400	----	----	3,446	20,710	816,711	29,845	3.7
Michigan	1,505,111	1,359,665	[17] 1,359,665	(9)	145,446	133,863	4,078	2,529	74	18	(10)	(10)	(10)	5,981	----	1,373,676	131,435	9.6
Minnesota	822,598	704,437	704,155	282	118,161	29,704	2,074	2,490	55	10	4,723	----	----	(18)	----	783,627	38,971	5.0
Mississippi	226,286	173,214	170,388	2,826	53,072	2,078	234	1,585	18	1	(10)	(10)	(10)	2,478	----	205,890	20,396	9.9
Missouri	835,895	701,894	701,438	456	134,001	31,055	1,712	2,222	12	16	2,079	----	8	2,105	----	809,615	26,280	3.2
Montana	173,892	134,729	133,811	918	39,163	2,812	457	1,731	20	----	[12] 1,846	[12] 36	----	986	----	167,150	6,742	4.0
Nebraska	412,726	350,833	350,569	264	61,893	30,091	985	1,166	9	2	2,339	----	40	1,600	2,718	413,787	−1,061	−0.3
Nevada	40,655	32,563	32,384	179	8,092	1,262	113	565	17	1	610	38	7	88	109	38,509	2,146	5.6
New Hampshire	125,939	99,983	99,983	(9)	25,956	4,471	963	649	16	1	600	----	----	594	----	122,236	3,703	3.0
New Jersey	994,497	861,795	856,423	5,372	132,702	6,692	4,674	2,653	75	9	9,490	----	567	2,895	----	943,412	51,085	5.4
New Mexico	118,106	90,833	90,322	511	27,273	2,660	337	1,905	99	----	905	153	40	368	----	108,729	9,377	8.6
New York	2,561,703	2,233,695	2,233,695	----	328,008	36,213	10,230	5,859	542	108	[12] 25,006	[12] 896	[12] 1,121	5,369	----	2,453,542	108,161	4.4
North Carolina	525,350	449,897	449,201	696	75,453	40,129	1,449	2,482	83	2	(18)	(18)	(18)	(18)	----	504,517	20,833	4.1
North Dakota	173,188	141,104	141,018	86	32,084	816	262	719	9	----	624	----	----	601	485	167,241	5,947	3.6
Ohio	1,876,132	1,695,648	1,695,648	(9)	180,484	119,269	8,194	2,595	117	16	18,881	829	----	4,612	28,007	1,777,048	99,084	5.6
Oklahoma	547,263	448,588	446,083	2,505	98,675	26,471	1,143	2,472	96	13	6,456	----	----	3,468	----	531,914	15,349	2.9
Oregon	360,348	299,689	298,971	718	60,659	(9)	1,559	2,457	33	3	4,289	----	----	715	1,093	332,729	27,619	8.3
Pennsylvania	1,984,821	1,738,797	1,733,014	5,783	246,024	26,889	10,864	4,187	84	28	17,327	480	1,349	33,126	----	[19] 1,857,366	127,455	6.9
Rhode Island	167,586	148,583	148,145	438	19,003	571	724	466	17	1	1,325	22	105	384	----	159,140	8,446	5.3
South Carolina	296,224	250,820	250,623	197	45,404	5,716	785	1,510	13	6	3,750	----	178	814	1,679	278,829	17,395	6.2
South Dakota	184,743	155,948	155,856	92	28,795	20,121	419	1,177	16	3	1,062	117	11	670	----	186,480	−1,737	−0.9
Tennessee	400,384	341,648	341,186	462	58,736	(9)	1,473	1,993	174	1	6,900	----	----	595	----	380,792	19,592	5.1
Texas	1,552,114	1,237,348	1,236,440	908	314,766	47,162	3,667	5,228	191	150	14,256	1,052	372	3,725	----	1,478,124	73,990	5.0
Utah	126,692	105,571	104,922	649	21,121	953	439	1,631	50	14	1,894	171	85	586	----	116,816	9,876	8.5
Vermont	88,958	79,606	79,501	105	[20] 9,352	1,706	491	806	13	----	(10)	(10)	(10)	595	----	84,155	4,803	5.7
Virginia	440,713	371,708	371,708	----	69,005	6,730	1,880	3,267	70	66	[12] 5,404	[12] 150	[12] 92	5,689	----	417,463	23,250	5.6
Washington	535,483	450,906	450,143	763	84,577	16,031	2,055	2,961	39	24	6,872	297	163	1,473	3,488	499,760	35,723	7.1
West Virginia	290,837	246,162	245,440	722	44,675	3,164	1,161	1,075	6	----	4,860	167	71	14,244	----	280,015	10,822	3.9
Wisconsin	854,374	713,166	712,510	656	141,208	5,411	3,240	2,165	21	6	8,149	152	266	2,626	----	835,178	19,196	2.3
Wyoming	81,837	64,459	64,459	----	17,378	10,099	281	1,155	35	14	627	93	----	361	----	76,603	5,234	6.8
District of Columbia	184,119	165,257	163,886	1,371	18,862	2,152	708	1,419	22	60	[21] 2,094	94	170	3,803	----	181,319	2,800	1.5
At large	----	----	----	----	----	----	----	2,432	32	7	----	----	----	----	----	----	----	----
Total	29,705,220	25,449,924	25,405,728	44,196	4,255,296	1,019,985	104,686	104,792	3,278	1,069	231,280	7,606	7,421	150,498	67,664	28,165,550	1,539,670	5.5

[1] Registration periods ending not earlier than November 30 and not later than January 31 are considered as calendar-year periods. In the case of States in which the registration period is definitely removed from the calendar year, registration figures were obtained for the calendar-year period.

[2] Wherever possible publicly owned vehicles and vehicles not for highway use have been eliminated from these columns.

[3] A complete segregation of motorbusses from other vehicles is not available. The figures given represent common-carrier busses in most cases, although in some States contract busses and contract school busses are included. In a number of cases city busses are not included, rural and interurban carriers only being given. Where no busses are tabulated the busses are included with automobiles, except as otherwise noted.

[4] Figures for trailers and semitrailers are as reported. Apparent inconsistencies are due to the fact that some States require the registration of tourist trailers, light-work trailers, and similar vehicles, whereas other States register only freight-carrying trailers and semitrailers.

[5] Data on Federal vehicles obtained through agency of Procurement Division, Department of the Treasury.

[6] Data on State, county, and municipal vehicles are incomplete in many cases. Some States give State-owned vehicles only; others exclude from registration certain classes, such as fire apparatus and police vehicles.

[7] Figures include new-car, used-car, and motorcycle dealers' registrations, and in some cases wreckers' and repairers' registrations. Data on dealers' extra plates incomplete, although extra plates are apparently included with dealers' registrations in some cases.

[8] Revised totals for 1936 registrations, resulting from revisions in two States, are as follows: automobiles, 24,137,423; motorbusses, 40,788; motortrucks, 3,987,339; trailers, 868,398; motorcycles, 97,063.

[9] Included with motortrucks.

[10] Included with private and commercial registrations.

[11] 1936 registrations revised to include 5,009 farm trucks licensed for limited highway use, which were omitted from table of State motor-vehicle registrations in 1936, previously issued. Revised motortruck registrations, 65,067.

[12] Includes unknown number of Federal vehicles.

[13] Includes 59,200 light trailers registered without charge.

[14] Trailers of 1,000 pounds capacity or more prohibited on highways, although permitted in cities under city licenses. Tractor-semitrailers registered as motortrucks. Light trailers permitted but not registered.

[15] Includes light trailers and commercial semitrailers. Commercial full trailers included with motortrucks.

[16] Of these vehicles approximately 1,400 are included with private and commercial registrations.

[17] Taxicabs included with motortrucks.

[18] Not reported.

[19] 1936 registrations revised to eliminate publicly owned vehicles, reissues, and exchange transfers, which were included in figures given in table "State motor-vehicle registrations, 1936." Revised figures as follows: Automobiles, 1,615,955; motorbusses, 5,577; motor trucks, 235,834; trailers, 23,776; motorcycles, 10,540.

[20] Trucks under 1,500 pounds capacity included with passenger cars.

[21] Includes 444 automobiles of the diplomatic corps.

STATE MOTOR-VEHICLE RECEIPTS, 1937

(Compiled for calendar year from reports of State authorities [1])

State	Total receipts, registration and other fees	Motor-vehicle registration fees					Registration fees, other vehicles		Total registration fees, all vehicles	Miscellaneous receipts								
		Total [2]	Passenger motor vehicles			Motor trucks, tractor trucks, etc.	Trailers and semi-trailers	Motor-cycles		Total	Dealers' licenses and plates	Operators' and chauffeurs' permits	Certificates of title	Fines and penalties	Transfer or reregistration fees	Other receipts	Unclassified refunds	Estimated service charges, local collectors [4]
			Total	Automobiles (including taxicabs)	Motor-busses [3]													
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	4,439	3,979							3,979	460	(5)	261		189		11	−1	
Arizona	1,113	807	384	355	29	423	55	1	863	280	6	33	101	2	23	112		
Arkansas	3,241	2,553	1,185	1,116	69	1,368	416	1	2,970	271	75	176		1	10	6		
California [6]	24,003	21,172	16,645	16,645	(7)	4,527	979	57	22,208	1,795	65	121		255	1,250	101		
Colorado	2,603	2,051	1,557	1,557		[8] 494	29	2	2,082	521	12	150	197	33	94	10	−5	
Connecticut	6,691	4,247	2,820	2,691	129	1,427	22	6	4,275	2,416	107	1,675		217	141	285	−9	
Delaware	1,262	870	596	596	(7)	274	30	1	901	301	7	132	43	75	19	25		
Florida	6,196	5,597	3,895	3,729	166	1,702	236	6	5,839	357	23	9	220		1	24	−28	108
Georgia	2,368	1,875	1,163	1,098	65	712	125	3	2,003	365	9	339			8	9		
Idaho	2,498	2,295	1,647	1,620	27	648	34	3	2,332	166	29	125				12		
Illinois	21,430	19,455	13,563	13,563	(7)	5,892	170	15	19,640	1,790	93	339	559	8	674	115	−28	
Indiana	9,827	7,768	5,898	5,800	98	1,870	329	7	8,104	1,723	51	628	438		499	68	−197	236
Iowa	11,918	10,888	8,213	8,213		2,675	86	8	10,982	936	52	565		2	213	104		
Kansas	4,537	3,636	2,773	2,773	(7)	863	75	4	3,715	822	31	418	317		41	15		
Kentucky	5,125	2,706	1,534	1,451	80	1,172	(9)	2	2,708	2,417	25	498		1	206	[10] 1,485		202
Louisiana	4,710	4,052	2,807	2,746	61	1,245	569	4	4,625	85	13	87		11		2	−28	
Maine	3,865	2,932	2,172	2,159	13	760	[11] 57	5	2,994	871	55	564		29	164	59		
Maryland	5,577	3,497	2,880	2,728	152	617	115	7	3,619	1,958	48	703	[12] 802	298	131	33	−57	
Massachusetts	6,875	4,221	2,815	2,684	131	1,406	23	2	4,246	2,629	72	2,551		6		68	−68	
Michigan	22,085	18,514	13,025	[13] 13,025	(7)	5,489	1,015	15	19,544	2,541	79	955	925		395	187		
Minnesota	8,867	8,309	6,379	6,232	147	1,930	258	6	8,573	294	54	125		89		26		
Mississippi	2,248	2,198							2,198	50	2			44		4		
Missouri	9,638	8,184	6,842	6,842	(7)	1,342	129	9	8,322	1,316	83	460	454	2	305	16	−4	
Montana [14]	1,532	1,257	894	871	23	363	9	1	1,267	265	19	116	32	43	46	9		
Nebraska	2,603	1,980	1,239	1,218	21	741	39	2	2,021	582	24	395			136	34	−7	
Nevada	282	267	163	162	1	104	10	1	278	4	3			(*)	11	3	−13	
New Hampshire	2,347	2,067						5	2,072	275	18	164		66		27		
New Jersey	19,271	13,601	9,572	9,269	303	4,029	262	9	13,872	5,399	72	3,814	599	202	664	61	−13	
New Mexico	1,544	1,421	929	888	41	492	25	1	1,447	97	10			2	28	63	−6	
New York	52,901	42,947	31,114	31,114		11,833	556	47	43,550	9,351	212	6,902		665	1,268	304		
North Carolina	8,855	8,546	5,432	5,377	55	3,114	(7)	6	8,552	303	(5)	142	179	7			−25	
North Dakota	1,581	1,407	1,067	1,057	10	340	3	2	1,412	169	18	90	61			1	−1	
Ohio	25,635	22,747	14,612	14,612	(7)	8,135	1,183	38	23,968	1,667	147	763			374	41		342
Oklahoma [15]	5,584	4,131	2,535	2,458	77	1,596	195	4	4,330	1,254	48	280	335	15		258		318
Oregon	3,378	2,617	1,534	1,495	39	1,083	(7)	5	2,622	756	28	452	214	32		30		
Pennsylvania	38,332	27,747	19,061	18,451	610	8,686	383	30	28,160	10,172	385	5,491	2,241	280	1,384	485	−94	
Rhode Island	2,790	2,194	1,712	1,670	42	482	2	3	2,199	591	17	417			100	57		
South Carolina	1,690	1,156	725	725		431	56	1	1,213	477	20	387		25	25	38	−18	
South Dakota	1,650	1,572	1,352	1,349	3	220	28	1	1,601	49	16		25			8		
Tennessee	4,233	3,910							3,910	323	(5)	295		19		10	−1	
Texas	19,684	18,308	11,733	11,583	150	6,575	482	16	18,806	878	46	309			377	146		
Utah	1,049	865	500	500	(7)	365	29	1	895	154	15	18	81	5	26	10	−1	
Vermont	2,410	2,036	1,422	1,408	14	[16] 614	21	4	2,061	349	26	282			55	5	−19	
Virginia	6,153	5,449	3,999	3,999		1,450	57	6	5,512	641	61	87	275		186	32		
Washington	4,402	2,530	1,427	1,427		1,103	212	6	2,748	1,654	12	1,252	222	(*)		30		138
West Virginia	6,162	4,706	3,640	3,619	21	1,066	60	5	4,771	1,391	53	202	[17] 1,043		69	24		
Wisconsin	12,984	12,281	9,234	8,940	294	3,047	353	15	12,649	335	38	41			242	14		
Wyoming	597	549	357	357		192	31	1	581	16	8				7	1		
District of Columbia	878	188	165	164	1	23	2	1	191	687	5	223	92	329	35	3		
Partial totals [18]			223,211	220,339	2,872	92,920	8,750	375										
Full totals	399,613	328,285							337,410	62,203	2,322	33,036	9,458	2,955	9,207	4,504	−623	1,344

[1] Registration periods ending not earlier than November 30 and not later than January 31 are considered as calendar-year periods. In the case of States in which the registration period is definitely removed from the calendar year, data on receipts were obtained for the calendar-year period. Stars (*) indicate receipts less than $500.

[2] Segregation of registration fees by type of vehicle was not made in Alabama, Mississippi, New Hampshire, and Tennessee. For these States the total motor-vehicle registration fees include those of trailers and motorcycles, except in the case of New Hampshire, for which motorcycle fees were reported separately. Dealers license fees in Alabama and Tennessee are also included.

[3] The figures for registration fees of motorbusses are incomplete (see preceding table, note 3). Where no fees are tabulated, the fees of busses are included with those of automobiles, except as otherwise noted.

[4] In a large number of States service charges are collected or deducted by the county or local officers who issue registrations. In the majority of cases these charges are included in the registration and other fees as listed. The amounts shown in this column are estimates of service charges collected and retained by local officials and not reported elsewhere in the table.

[5] Included with motor-vehicle registration fees.

[6] Registration fees include proceeds of vehicle license fees, $10,728,000, imposed in addition to the regular registration fees, the proceeds of which were $11,480,000.

[7] Included with fees of motortrucks.

[8] Fees of 22,296 light trucks included with those of passenger vehicles.

[9] Trailers of 1,000 pounds capacity or more prohibited on highways, although permitted in cities under city licenses. Tractor-semitrailers registered as trucks. Light trailers permitted but not registered.

[10] Includes proceeds of motor usage tax, $1,422,000, imposed at 3 percent of retail price on all motor vehicles at time of first registration.

[11] Fees of light trailers and commercial semitrailers only. Fees of commercial full trailers included with those of motortrucks.

[12] Includes proceeds of 1 percent title tax on motor vehicles, $587,000, in addition to regular title fees.

[13] Fees of taxicabs included with those of motortrucks.

[14] Registration fees are collected by counties, and State does not maintain complete record. Figures given are estimates supplied by State.

[15] Receipts do not include proceeds of 2 percent motor-vehicle excise tax, which is imposed as part of a general sales tax. Proceeds of this tax in 1937 were $1,244,000.

[16] Fees of trucks under 1,500 pounds capacity included with those of passenger cars.

[17] Includes proceeds of 2 percent title privilege tax, $863,000, in addition to regular title fees.

[18] Totals of columns for which fully classified totals were not available for all States.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JUNE 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 3,765,925	$ 1,700,138	70.7	$ 6,690,637	$ 3,340,190	272.8	$ 3,082,295	$ 1,536,555	135.8	$ 3,873,445
Arizona	2,504,264	1,730,523	115.6	1,669,555	1,264,931	91.9	168,163	110,953	11.3	1,799,834
Arkansas	3,127,464	3,098,036	188.7	1,110,835	1,100,169	75.9	14,040	12,962	4.0	4,320,542
California	7,255,457	3,911,392	158.4	11,139,156	5,864,481	204.0	3,305,292	1,756,799	80.1	2,209,182
Colorado	3,707,458	2,003,473	135.2	2,560,032	1,401,380	81.1	490,560	272,740	10.9	3,038,935
Connecticut	1,014,718	497,319	10.1	1,189,000	579,991	13.1	405,970	201,510	3.6	1,522,970
Delaware	534,818	263,575	21.0	499,605	249,615	8.5	817,877	404,920	15.2	1,184,277
Florida	1,487,243	740,764	38.0	2,705,351	1,352,676	55.0	2,538,156	1,269,078	35.8	2,724,162
Georgia	3,018,094	1,469,695	161.4	6,157,563	3,078,781	296.7	3,478,660	1,739,330	175.6	5,515,965
Idaho	2,775,318	1,605,670	211.8	1,544,315	914,117	143.8	1,151,688	686,653	49.4	1,305,659
Illinois	11,654,717	5,652,894	337.0	9,609,752	4,794,857	219.7	4,856,690	2,428,345	121.0	3,272,279
Indiana	6,336,691	3,111,645	153.1	5,164,872	2,576,390	131.3	2,933,620	1,466,440	61.6	2,596,312
Iowa	7,614,654	3,582,720	249.2	6,933,573	3,097,923	219.6	2,154,160	1,016,050	73.9	1,598,050
Kansas	4,712,215	2,313,656	259.8	4,348,427	2,174,144	167.2	5,178,900	2,545,771	737.0	3,576,379
Kentucky	3,038,093	1,507,265	95.2	5,189,644	2,594,822	167.7	3,278,186	1,626,351	92.0	2,255,884
Louisiana	582,531	286,501	15.0	11,793,628	2,304,315	54.8	2,032,266	900,022	36.6	2,516,382
Maine	2,275,114	1,120,950	62.4	2,511,280	1,253,210	57.7	649,164	324,581	12.0	672,327
Maryland	1,048,330	524,145	15.0	1,727,147	861,811	27.3	1,494,831	735,040	21.4	1,992,259
Massachusetts	4,423,400	2,211,667	20.3	2,690,859	1,345,426	10.6	302,491	151,244	2.7	2,847,916
Michigan	7,208,466	3,378,283	174.0	6,262,438	3,053,594	130.7	1,274,850	636,800	20.1	3,450,087
Minnesota	6,739,297	3,324,796	351.3	5,151,345	2,553,639	239.7	1,534,556	762,037	108.0	3,528,637
Mississippi	3,383,701	1,676,213	168.4	6,138,010	2,838,596	268.2	3,306,050	871,435	148.8	3,443,858
Missouri	9,243,761	4,514,211	490.9	5,005,396	2,473,036	137.3	4,001,940	1,673,031	153.0	3,963,961
Montana	4,410,403	2,466,168	323.5	1,726,038	970,524	70.4	274,393	154,344	8.7	4,462,564
Nebraska	3,370,938	1,657,038	356.6	5,802,086	2,892,572	519.0	4,065,158	1,397,582	189.0	2,785,460
Nevada	2,455,650	2,051,622	132.5	1,215,593	1,052,976	89.2	609,990	529,031	68.6	1,490,200
New Hampshire	451,364	221,487	7.1	1,041,513	518,292	19.0	270,427	134,435	7.0	1,172,082
New Jersey	1,941,364	953,171	19.0	2,852,361	1,424,918	21.2				2,776,553
New Mexico	5,362,486	3,286,151	365.7	1,535,623	1,036,718	118.5	1,169,118	713,038	120.8	1,134,063
New York	15,779,311	7,136,340	282.0	14,885,865	7,314,782	264.9	3,288,230	1,640,440	40.8	4,248,388
North Carolina	5,660,057	2,823,987	406.0	7,098,513	3,426,252	331.0	956,950	420,300	59.3	3,388,863
North Dakota	1,091,282	1,012,725	186.5	2,534,591	2,456,361	155.3	531,620	462,946	64.7	3,813,287
Ohio	5,722,429	2,781,464	73.7	8,554,132	4,246,620	91.4	3,681,450	1,840,514	25.8	7,760,747
Oklahoma	4,223,945	2,200,586	220.2	4,251,156	2,199,273	140.3	3,420,061	1,798,378	139.3	3,387,789
Oregon	4,301,657	2,527,049	170.2	2,166,046	1,315,165	84.8	607,825	327,325	11.9	2,326,410
Pennsylvania	14,807,231	7,319,221	206.2	6,684,602	3,328,502	103.0	2,850,976	1,410,844	43.3	6,093,096
Rhode Island	1,198,028	570,474	11.3	930,542	465,271	13.6	240,060	120,030	2.4	1,158,499
South Carolina	4,226,732	1,750,353	287.0	5,321,595	2,359,135	226.6	1,221,955	552,090	68.5	1,866,336
South Dakota	2,542,631	1,433,833	235.2	3,386,532	1,875,710	328.9	1,478,650	825,520	103.8	3,442,676
Tennessee	2,352,823	1,171,703	90.3	5,438,714	2,719,357	167.6	663,460	331,730	16.7	4,938,008
Texas	16,139,393	7,983,089	1,076.7	12,198,721	6,006,923	675.5	3,644,192	1,745,951	213.5	8,602,515
Utah	1,743,705	1,210,506	150.3	1,085,080	776,620	99.6	433,670	307,023	36.8	1,721,289
Vermont	1,646,108	792,849	44.5	1,416,851	588,252	34.6	517,668	243,048	15.0	294,747
Virginia	4,797,348	2,342,899	181.0	5,005,036	2,501,242	163.5	2,871,056	1,434,678	89.5	1,119,840
Washington	2,773,133	1,439,605	79.6	4,381,676	2,307,870	71.9	716,429	352,654	17.9	1,350,405
West Virginia	1,630,364	837,810	45.4	1,786,146	1,189,183	53.3	795,956	505,778	19.3	2,518,310
Wisconsin	8,511,485	4,144,419	281.4	6,088,787	2,804,877	140.9	3,002,004	1,322,280	109.0	2,161,311
Wyoming	2,996,278	1,868,132	321.7	1,963,252	1,204,506	208.3	386,250	238,640	50.6	893,056
District of Columbia										
Hawaii	845,057	412,656	13.3	898,690	446,060	18.5	776,440	380,075	8.8	1,198,792
Puerto Rico				1,128,859	562,390	20.5				671,985
TOTALS	218,432,931	112,620,868	9,069.4	219,171,020	109,058,445	7,305.9	86,924,393	42,317,321	3,640.8	139,986,573

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JUNE 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 415,800	$ 207,800	23.8	$ 250,400	$ 123,650	20.1	$ 721,522
Arizona	$ 98,773	$ 66,705	9.9	196,056	129,008	11.4	92,929	63,940	7.2	463,534
Arkansas				13,126	6,563					857,545
California	127,081	73,766	37.2	647,482	367,862	43.9	834,528	453,336	37.0	1,023,734
Colorado				408,026	225,303	19.3	318,330	176,990	14.1	519,275
Connecticut							47,530	23,750	.9	294,528
Delaware										246,875
Florida				20,122	10,061					664,791
Georgia	38,120	19,060	3.2	308,558	154,279	38.6	322,700	161,170	37.9	943,117
Idaho	304,816	150,274	43.1	285,338	131,709	12.4	139,383	53,490	32.7	286,300
Illinois	79,800	39,900	3.3	958,232	479,116	78.9	1,421,900	710,950	114.7	836,849
Indiana				356,500	134,200	40.2	912,853	416,800	87.7	691,438
Iowa										1,298,449
Kansas	24,921	12,458	24.4	43,670	21,835	.5	177,100	88,550	26.3	1,208,618
Kentucky	246,502	122,573	105.1	532,214	148,425	50.6	1,259,457	390,987	131.9	267,692
Louisiana				21,670	10,835		459,044	172,946	33.4	540,355
Maine	228,304	113,833	15.4	219,636	109,818	14.7	194,600	97,300	10.2	121,455
Maryland				6,264	3,132					409,344
Massachusetts				5,300	2,650					643,750
Michigan				23,362	11,681		316,000	158,000	26.3	1,365,858
Minnesota				351,318	156,010	32.0	55,560	25,417	13.4	1,198,153
Mississippi										888,927
Missouri	972,621	468,046	246.5	325,020	161,215	36.8	343,610	125,900	38.6	775,403
Montana				13,983	7,865					1,027,170
Nebraska	93,312	46,656	8.9	288,740	144,370	46.1	391,660	190,656	61.8	662,426
Nevada	178,752	153,067	26.5	229,813	199,011	23.5	182,643	158,316	30.6	134,117
New Hampshire				101,176	50,152	1.8	204,889	74,848	3.9	121,875
New Jersey							119,020	55,855	1.9	616,918
New Mexico				541,394	330,193	30.7				476,579
New York	230,525	114,312	19.0	2,161,360	1,080,680	144.5	213,400	106,700	3.7	1,171,081
North Carolina	203,170	101,472	46.4	678,440	339,220	75.3	308,373	123,750	22.6	619,115
North Dakota							57,640	30,870	9.0	756,382
Ohio				184,400	92,200	3.8	11,100	5,000	.9	1,734,441
Oklahoma				16,888	8,986		543,260	279,022	49.8	896,907
Oregon	133,066	75,035	36.4	261,775	158,752	29.3	239,493	136,300	29.8	458,070
Pennsylvania	58,660	29,330	5.9	955,311	458,307	70.5	1,087,460	530,830	59.2	1,126,751
Rhode Island	122,420	58,984	3.3	85,070	42,535	3.2	48,090	23,481	2.9	121,875
South Carolina				465,248	195,862	52.7	517,850	221,754	59.0	261,064
South Dakota				11,300	6,250					816,436
Tennessee				206,546	103,273	8.8	159,520	79,760	6.5	876,457
Texas	49,200	24,545	25.6	1,313,811	604,182	166.9	1,303,513	581,575	189.8	1,938,768
Utah	36,203	21,346	4.8	242,365	134,610	22.4	182,065	98,630	12.2	317,130
Vermont	76,012	37,335	4.3	200,056	87,853	11.6				121,687
Virginia	55,300	27,650	11.7	458,517	227,043	43.7	476,320	190,970	22.1	473,986
Washington	117,776	61,874	18.0	334,985	176,078	23.7	459,996	242,100	40.5	311,791
West Virginia				208,000	104,000	16.5	118,200	59,100	9.4	386,574
Wisconsin	30,710	15,250	6.7	564,015	266,305	22.7	194,099	86,470	8.1	859,453
Wyoming	75,809	46,849	7.2	378,420	233,820	43.8	61,500	38,000	5.1	313,373
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico				244,000	121,950	13.7				124,925
TOTALS	3,581,853	1,880,320	712.8	15,339,557	7,673,124	1260.7	14,026,015	6,557,163	1,261.2	33,211,613

PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T . . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . . Bibliography on Highway Safety.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JUNE 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			
			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 16,100	$ 16,100	1			$ 541,798	$ 541,724	7			$ 125,810	$ 125,810	2		6	$ 1,317,985
Arizona						4,718	4,718									625,495
Arkansas						279,639	278,482	7			110,000	110,000	1			1,370,287
California						1,200,958	1,200,383	3	4		144,157	144,157	1			2,355,669
Colorado	1,880	1,880			1	35,728	35,728	1			14,251	10,491			4	1,241,823
Connecticut																844,490
Delaware											77,270	77,270			23	416,480
Florida						10,616	10,616				178,800	178,800	1			1,216,381
Georgia						18,346	18,346									2,399,041
Idaho						100,167	99,994	1								722,876
Illinois	19,500	19,500		2		669,675	669,675	3			569,500	569,500	7		1	3,965,468
Indiana	165,790	165,790	2			791,590	687,313	5	5		720,260	683,600	1	1	86	1,041,793
Iowa	226,215	215,829	4	3	1	914,875	864,300	11	1	4	131,620	122,500	1		17	1,571,017
Kansas	6,290	6,290			3	471,660	471,660	10		3	500,719	500,719	7		3	1,600,424
Kentucky						145,000	145,000	1			283,724	283,724	2		31	1,385,321
Louisiana						23,648	23,648				273,828	273,680	6			1,279,743
Maine						188,647	188,647	2	1		248,140	248,140	3	1	1	254,416
Maryland						64,586	64,586	1								962,247
Massachusetts						70,420	70,420		1		162,480	162,480	1			1,680,388
Michigan						568,152	568,152	3		45	869,200	869,200	9	1		1,847,833
Minnesota	235,218	235,218	1		16	547,766	547,766	2	4	1	98,650	98,650		2		1,775,066
Mississippi						252,700	252,700	3			103,300	103,300	1			1,228,151
Missouri						236,070	236,070	4			149,140	149,140	2			2,640,043
Montana						365,654	360,772	4			276,614	276,614	2			687,085
Nebraska						311,783	311,783	11			13,880	13,880	2			1,438,747
Nevada	35,109	35,109		1		93,878	93,878		2	1	55,883	55,883		1	1	308,880
New Hampshire						65,482	65,175		1							428,575
New Jersey						210,005	204,779	2	1							1,765,478
New Mexico						122,441	122,441	4								729,050
New York						1,643,901	1,642,100	6	4	2	650,068	642,518	2	3		4,485,311
North Carolina	43,180	43,180	(•)			417,800	417,800	3	2		108,120	108,120	1	2		1,918,474
North Dakota						534,046	534,046	1	1		62,630	62,630	1	1		982,545
Ohio						32,120	32,120				267,110	267,110	4			3,929,938
Oklahoma						17,343	17,343									2,264,166
Oregon	178,890	178,890	3			293,683	293,683	1	1		32,021	31,372		1		650,276
Pennsylvania						224,030	208,824	1			75,086	75,086	1			5,448,625
Rhode Island						223,897	223,897	1								269,853
South Carolina						38,473	16,137		1		387,344	387,344	2	1	22	1,085,621
South Dakota	50,960	50,960			15	120,838	120,838	1	1		98,575	98,575	2		10	1,098,084
Tennessee						14,381	14,381									1,877,652
Texas	58,035	57,800	3			38,797	38,100	2			950,388	942,903	9	2		4,360,065
Utah	71,000	71,000	2		2	105,073	105,073	2			22,960	22,960	1		4	444,955
Vermont	14,268	14,233			5	210,288	205,288	6	1		28,850	24,570		1	1	249,659
Virginia	80,710	80,710	2	3		298,715	298,715	12	2		286,843	286,843	2		3	1,210,472
Washington	134,774	134,774	2	1		414,742	412,642	5	2		252,855	252,855	3	1	4	718,301
West Virginia						303,939	302,919	2			144,335	144,335	2		3	876,844
Wisconsin						947,357	936,787	8			207,138	175,000	2			1,361,722
Wyoming						144,884	144,884		2		14,530	14,530			6	517,316
District of Columbia	168,320	168,320		1												325,431
Hawaii											197,540	197,540	3	1		296,210
Puerto Rico						61,900	61,550	2			159,001	157,920	2			510,830
TOTALS	1,506,239	1,495,583	20	11	43	14,392,209	14,165,883	138	37	56	9,052,620	8,949,749	86	19	226	73,982,602

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 6 AUGUST 1938

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 6 August 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

STUDY AND STANDARD OF QUALITY OF A ROAD SURFACE[1]

FROM THE VIEWPOINT OF (A) ITS SLIPPERINESS OR RUGGEDNESS, AND ITS RESISTANCE TO SKIDDING; (B) ITS PROPERTY OF REFLECTING OR ABSORBING LIGHT (UNDER ARTIFICIAL LIGHTING)

By **R. H. BALDOCK**, Chief Engineer, Oregon State Highway Commission, Principal Reporter

GENERAL CONCLUSIONS

1. To afford reasonable protection against the dangers of skidding, a road surface should have a static or side-skid friction coefficient of 0.4, or higher, at 40 miles per hour.
2. Pavements with a coarse sandpaper or slightly rough texture are more nonskid than pavements having a rough, knobby texture.
3. As the speed of the vehicle increases, particularly on wet surfaces, the coefficient of friction decreases.
4. The coefficient of friction of wet surfaces of the bituminous type increases with an increase of hardness of the asphalt or tar binder.
5. The coefficient of friction of wet bituminous surfaces varies through a wide range. The coefficients of the high-type, nonskid asphaltic pavements and bituminous macadam surfaces are consistently higher than the coefficients of other wet surfaces. However, unusually low coefficients are obtained on bituminous surfaces when such surfaces are covered by a skin of free asphalt.
6. The coefficients of wet portland cement concrete surfaces are from 15 to 50 percent lower than those of high-type asphaltic surfaces under similar conditions of test.
7. The lower coefficients of friction of wet portland cement concrete surfaces, in comparison with present high-type bituminous macadam and asphaltic concrete surfaces under similar conditions, are unquestionably caused by the film of water retained on the surface of portland cement concrete pavements of current design. Tests indicate that the presence of the water film on the portland cement concrete surfaces is the probable cause of the reflection or glare from approaching headlights as evidenced on this type of pavement in comparison with the noticeable absence of glare afforded by the rough-textured surfaces of modern asphaltic concrete and bituminous macadam.
8. Changes in the design of portland cement concrete pavement surfaces to afford larger grooves to permit the rapid run-off of the water are necessary to increase skid resistance and to reduce the headlight glare of this type of pavement.
9. The larger of two tires of the same tread design will have the greater coefficient of friction. This substantiates the theory that the total frictional resistance of tires on road surfaces is equal to the true frictional resistance of the two materials plus the mechanical resistance which increases with the contact area of the tire.
10. On wet surfaces the squeegee action of the edges of new-tread tires provides a more intimate contact with the area of the road surface than is possible with smooth-tread tires and thus reduces the lubricating effect of the water.
11. Dry, light-colored pavements are from 2 to 4 times as luminous as dry, dark-colored pavements. No pavement has much luminosity when wet.
12. Smooth-textured pavements reflect more light than pavements having rough-textured surfaces.
13. Both smooth pavements and pavements having a fine, sandpaper texture are covered with a film of water, when wet, and this film is a very effective reflecting surface. Coarse-textured surfaces having pits or indentations of from ⅛ to ¼-inch in depth break up the water film and therefore reflect light in considerably lesser degree.
14. Reflective glare from wet surfaces is in the main a function of the texture of the surface and the least glare is given by those surfaces which are so deeply indented that water is not held by surface tension but drains off with sufficient rapidity to prevent the flooding of the entire surface and the formation of a water film to act as a mirror and reflect the lights of approaching vehicles.
15. For a maximum of skid resistance and a minimum of glare, it is necessary to construct pavements having surface indentations or grooves deep enough to prevent the surface tension of water from holding a film of water on the surface. Moreover, the small particles of rock or sand should stand above and screen off the light from the water running in the indentations or grooves above described.
16. The surface texture of the pavement should be as fine as possible to accomplish the above objective inasmuch as a coarse, knobby texture reduces skid resistance.
17. Water film, when present on the surface of a pavement, acts not only as a lubricating medium between the tire and the pavement but also as a mirror to reflect light and cause dangerous glare.

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[1] Report for the United States to the Eighth International Road Congress held at The Hague, June 19-July 2, 1938.

206332

A. SLIPPERINESS OR ROUGHNESS AND RESISTANCE TO SKIDDING

Statistics collected and published by the Travelers Insurance Company of America indicate that about 1,500 persons are killed annually and that 30,000 persons are injured annually in the United States of America, by accidents reported as due to skidding on slippery road surfaces. While many of these accidents were in all probability caused by attempted driving at excessive speed, it must be borne in mind that, to the average motorist, skidding is a menace which causes uneasiness and discomfort even when no accident results. It is quite probable that many such accidents occurred on surfaces so slippery as to afford practically no friction between the tire and the pavement.

Any road surface used by motor vehicles should have sufficient roughness of surface texture to provide that friction between the tires of automotive vehicles and the road surface requisite for the safe starting, steering, and stopping of such vehicles under all reasonable conditions of operation. No road surface should be so slippery that skidding of the vehicle can occur when the vehicle is being operated in a reasonable and proper manner. Investigations have shown that the margin of safety for high-speed driving is small even on dry surfaces where high coefficients of friction are available.

The knowledge of the coefficient of kinetic friction between automobile tires and roadway surfaces now extant is due almost entirely to the work of Professor R. A. Moyer and his associates as reported in Bulletin 120 of the Iowa State College. This report will deal with the findings of Professor Moyer and his associates from numerous tests of the skid-resisting properties of various types of surfaces. It will also make use of the reports and observations contributed by various highway departments and other organizations and individuals.

DEFINITIONS

Coefficient of friction.—The coefficient of friction is the ratio between the force parallel to the road surface and the normal pressure of the tires upon the road when skidding is impending or actually taking place. If skidding is impending, the coefficient is called the coefficient of static friction, while if the movement is actually taking place the coefficient is called the coefficient of kinetic friction. Static friction will control skidding when a vehicle has just started and also the straight skidding in the direction of travel when the brakes are on the point of locking the wheels against rotation. Kinetic friction will control straight skidding in the direction of travel when the wheels are locked, and sidewise skidding in the direction normal to the line of travel with wheels either locked or rotating.

If a great force is necessary to cause skidding, the coefficient will be high, while if a slight force causes skidding the coefficient will be low. In other words, a pavement having a texture which enables the tires to grip the surface will offer great resistance to skidding and will have a high coefficient of friction. It therefore follows that highway surfaces are skid-resistant in the order of the ascending curve of the coefficient of friction.

Integrating dynamometer.—The integrating dynamometer is an instrument for measuring the force exerted by the vehicle when skidding straight ahead after the application of the brakes, or the amount of skid force exerted by the vehicle as it tends to skid sideways. The dynamometer is simple and rugged in construction, measures accurately to within 10 pounds forces ranging from 100 to 2,000 pounds, and its accuracy is not changed by the vibration set up in the test-trailer. This instrument was used in the tests hereinafter described.

TESTS BY PROFESSOR MOYER

Road tests to determine the slipperiness of various road surfaces under various surface conditions, as wet, dry, and snow- or ice-covered, were conducted by the Iowa Engineering Experiment Station in 1924 and again in 1927, under the direction of T. R. Agg. In 1932, the study was renewed and has been carried on continuously since that time under the direction of R. A. Moyer.

In the more recent work, all of the more important phases of skidding relating to highway safety were carefully investigated and analyzed. The resistance to skidding as measured by the coefficient of friction between the tires and the road surface was determined for 25 different types of surfaces. Tests were conducted with new tires of unworn tread design and with similar tires on which the tread had been worn smooth. They were conducted on wet and dry surfaces for both straight-ahead and sidewise skidding, at speeds ranging from 3 to 40 miles per hour, using a two-wheel trailer unit. Tests were made on pavements of brick, asphaltic concrete, portland cement concrete, on surfaces of penetration macadam using asphalt, tar, or road-oil binders, on surfaces treated with both tar and road oil, on gravel and cinder road surfaces, and on bridge floors surfaced with asphalt plank, steel plates, and hardwood plank. Tests were also made on mud-, snow-, and ice-covered surfaces both with and without tire chains.

On the basis of the results of braking-effort tests made with 2,134 cars and an analysis of the frictional requirements when braking, minimum safe-stopping distances were established.

Probably the most important phase of this investigation as far as this report is concerned is that which deals with the tests of the various road surfaces. It is now generally recognized that roads must be smooth-riding and free from large surface inequalities, yet skid-resistant to permit safe operation at the high speeds demanded today and the higher speeds which may be required by traffic in the future.

The Moyer tests indicated that skid-resistance can be provided in road surfaces of any type, and that the greater hazard, caused by snow, ice, and mud, can be materially reduced by maintenance. The tests indicated that practically all roads in the dry condition can provide the requisite friction with a liberal amount to spare for ordinary driving demands or even for emergency demands. Many surfaces in wet condition were found to be almost as safe as the same surfaces in dry condition, but the tests indicated that there is a large mileage of surfaces in the United States today which are dangerously slippery when wet.

In the earlier investigations of the skid-resistance of pavements, tests were conducted at low speeds at which the large uniform tractive forces required of the towing vehicle could be easily provided. In the more recent tests, the more variable effect of speeds ranging from 3 to 40 miles per hour was investigated. The results were startling because they indicated how slippery certain surfaces really are. On certain surfaces in wet condition, it was found that at 40 miles per hour the friction available is only ¼ to ⅕ as great as it is at 10 miles per hour. In fact, under certain conditions, these surfaces were found to be equally as slippery as snow- or ice-covered surfaces.

In theory, which was substantiated by the tests, the total frictional resistance to skidding is equal to the true frictional resistance between the two surfaces plus the resistance which results from a mechanical interlocking of the tire tread with the road surface. This mechanical resistance is brought about by the interlocking of minute particles of rubber with equally small projections in the road surface which tend to shear the rubber as the tire slides over the surface. In extreme cases skid marks are evidenced by the film of rubber left in the path of the skidding tire. The true frictional resistance is theoretically proportional to the normal load supported by the tire. The mechanical resistance, however, is dependent not only on the normal force or load on the tire, but depends also on the area of contact of the tire with road surface.

Surfaces which are smooth and which provide a large and intimate contact area for the tire, as indicated by tests, provide the greatest frictional resistance against skidding. Conversely, surfaces which have a rough knobby surface texture do not provide such an intimate contact, and therefore do not afford frictional resistances as great as those provided by the smooth surface with gritty finish.

On wet surfaces, the water serves as a lubricant if it cannot be squeezed or squeegeed from the surface as the tire rolls over it. At speeds above 25 to 30 miles per hour the tires fairly skim over the surface and it is far more difficult to remove the excess water and to overcome the lubricating effect. If the water is trapped between the tire and the road surface, skidding resistance drops off sharply. On wet surfaces, it is important to recognize that a large area of intimate contact is necessary to develop high frictional resistance; that small grooves should be provided in the road surfaces and in the tire to permit the excess water to escape; and that the aggregate on the surface should be hard and sharp, with many points of contact to grip the tire and prevent skidding. The tests revealed that highly polished or glazed surfaces are certain to be dangerous when wet. Examples of such types of surfaces are smooth steel plates, steel-troweled or highly polished concrete surfaces, and glazed-over asphalt surfaces.

The coefficients of friction of a considerable number of pavement surfaces when both wet and dry and at speeds of from 3 to 40 miles per hour were determined in the Moyer tests. Typical curves resulting from the tests are reproduced in figure 1; and in the following summary of the tests the average values from these curves are used. In figure 2, the results shown in figure 1 have been grouped so that those surfaces which give similar coefficients of friction under the conditions tested are shown by a single curve. Additional data relative to the effect of the condition of the tires are also shown in the lowest section of figure 1.

TESTS ON VARIOUS TYPES OF SURFACE

1. *Portland cement concrete road surfaces.*—Portland cement concrete surfaces of reasonably uniform standard exist in greater mileage than any other type of high-class road surfacing used in the United States today. While the skidding resistance of wet portland cement concrete pavement was found to be not as high as that of certain bituminous surface types, the skidding properties of portland cement concrete were found to be the most consistent for both wet and dry surfaces of all the types considered. The tests indicated the desirability of grooving the surface or of finishing with a canvas belt so as to bring the sharp sand grains to the surface and remove or break up the coating of cement paste which is responsible for the slippery condition certain to develop on a portland cement concrete pavement finished with steel trowels.

The cement paste develops a hard surface finish which is polished smooth by the wearing action of traffic. Excessive manipulation, vibration, or finishing of concrete pavement surfaces draws an excess of cement paste to the surface and makes a slippery surface. Proper brooming or belting will provide a skid-resistant surface. The coefficient of friction of wet portland cement concrete surfaces ranges from 0.4 to 0.9 for straight skidding and from 0.5 to 0.8 for sidewise skidding; the coefficient of friction of dry, rough-textured portland cement concrete surfaces ranges from 0.8 to 1.0 for straight skidding and from 0.9 to 1.0 for sidewise skidding. The ranges given cover the coefficients for both new-tread and smooth tires.

2. *Bituminous road surfaces.*—The frictional resistance of bituminous road surfaces, as measured in the tests, varied more widely than the frictional resistance of any of the other surfaces tested. This general class of surfaces has been generally thought of by the traveling public as dangerously slippery when wet. While the results of the skidding tests on several bituminous surfaces substantiated this belief, the majority of the bituminous surfaces, when tested in the wet condition, gave a frictional resistance appreciably higher than that obtained on any other type of surface. The tests by Professor Moyer substantiate the conclusions of many highway engineers that it is possible to build bituminous surfaces that will be as free from the dangers of skidding as any road surfaces in common use today. The majority of bituminous surfaces built recently in the United States are nonskid in character.

An examination of the bituminous surfaces found in the tests to be slippery revealed that they were covered with an excess of bituminous material. The surfaces which offered high resistance to skidding were covered with sharp, hard sand, or with hard and finely crushed rock particles held in place by the bituminous cement. Such surfaces had a sandpaper-like finish. The coefficient of friction for wet bituminous surfaces varied through a wide range of values. The coefficients for wet, high-type asphaltic surfaces were consistently higher than those for the other wet surfaces tested. However, unusually low coefficients were obtained on some bituminous surfaces, particularly on wet penetration macadam built with a soft seal coat.

It is quite apparent that the coefficient of friction increased with an increase in the hardness of the asphalt or tar binder used in the bituminous surfaces. In this connection it is pertinent to refer to the recent tendency in the practice of State highway departments in the western portion of the United States toward the use of softer binders to overcome cracking of the surface. It is the reporter's opinion that, by and large, the cracking that develops is a function of weak foundations and poor drainage, and that these factors should be corrected before hastily deciding to use softer asphalts. The tests by Moyer indicate that it is a sound practice to construct seal coats with cut-back or emulsified asphalts,

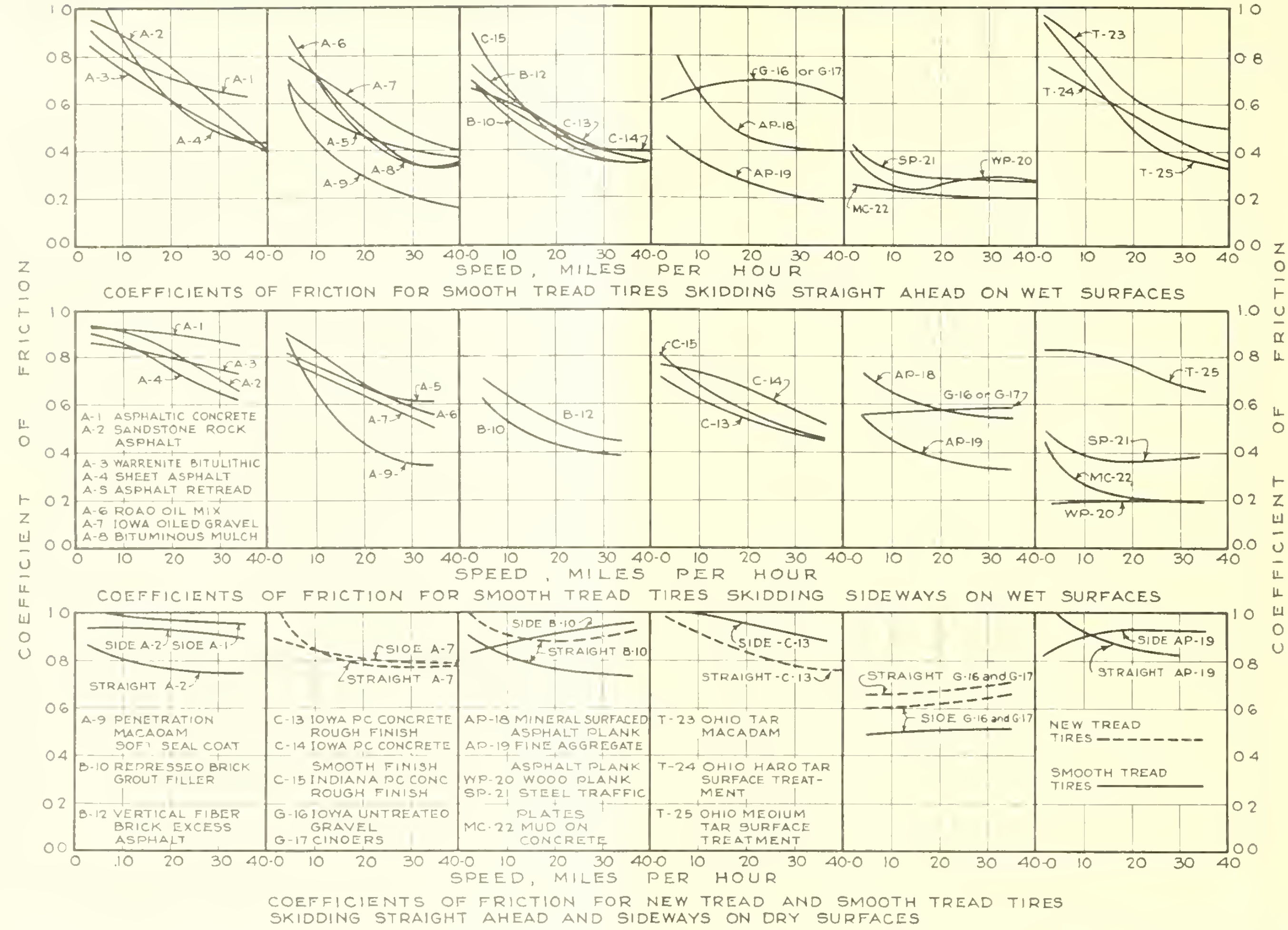


Figure 1.—Coefficients of Friction for New-Tread and Smooth-Tread Tires Skidding Straight Ahead and Sideways on Various Types of Road Surfaces.

small amounts of which are sufficient for the thorough coating of the cover aggregate. The thin film of asphalt surrounding the stone particles firmly binds them together and the absence of an excess of asphalt below the cover stone insures a granular surface.

Competent observers are of the opinion that, when hot asphalt is used for sealing, a cover stone of ¾-inch diameter should be employed to prevent flushing of the excess asphalt to the surface. Professor Moyer's experiments have thoroughly demonstrated that a sandpaper surface which takes the wear of traffic directly on the mineral aggregate is the kind of surface it is desirable to construct. Contrary to general observation, sheet asphalt pavement, properly constructed with a sandpaper texture, is quite resistant to skidding. It is, of course, necessary to eliminate the usual surface dusting coat. Since a slight excess of asphalt will destroy the sandpaper finish and produce a glazed surface, it appears necessary in bituminous pavement construction of both the premixed and penetration types to use a hard aggregate and an asphalt with a high melting point. This is particularly true for the top or wearing course. The requisite proportions between the aggregate and asphalt binder should be controlled by careful inspection during construction.

Nonskid asphaltic surfaces can be built either in accordance with the principle of maximum density or by the use of the more open mixes. The latter are perhaps easier to construct and more certain in their results. The methods of building the more open-textured mixes and penetrations are described more thoroughly hereafter in the report on the Oregon practice.

The coefficient of friction of dry bituminous pavement ranges from 0.7 to 0.9 for straight skidding and from 0.8 to 1.0 for sidewise skidding; the coefficient of friction of wet bituminous pavement ranges from 0.2 to 1.0 for straight skidding and from 0.3 to 0.9 for sidewise skidding. Again these ranges cover the use of both new and worn tires.

3. *Brick pavements.*—Brick surfaces, free from excess filler, were found in the Moyer experiments to have skidding resistance slightly lower than that of portland cement concrete surfaces. Brick surfaces with excess filler that exudes or bleeds out of the joints, covering the greater part of the surface with asphalt, were found to be as slippery as bituminous surfaces with seal coats containing an excess of bituminous material. This would be expected since the cause of the slipperiness is the same in both cases—the excess bituminous material on the surface. Recent developments in the manufacture of paving brick and in brick pavement construction have greatly improved the skid-resistant properties of brick pavements. De-aired, vertical-fiber brick with lugs should provide a wear-resisting, gritty finish. Where there is any difficulty in obtaining the gritty

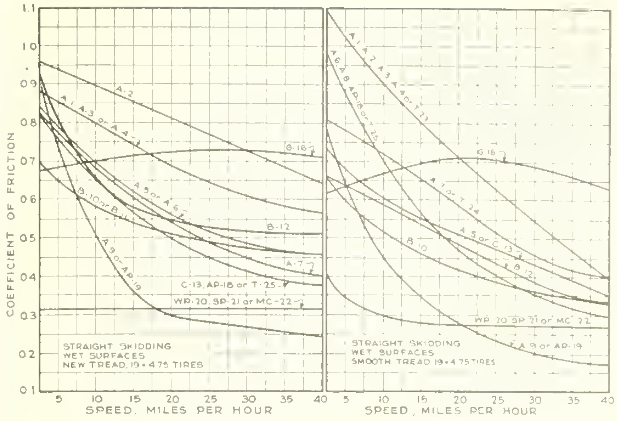
finish, it is possible to face the brick with carborundum or emery grit during manufacture and thereby provide a wear-resisting and skid-resisting surface which should be the equal of any road surface now in general use.

Bleeding of the joints has been practically eliminated by using a blended asphalt joint filler mixed with a mineral flour to provide a stable mix with a low coefficient of expansion. Before applying the joint filler, the brick surface may be sprayed with a separating agent consisting of a clacium chloride solution composed of 35 percent calcium chloride, 1 percent starch, and 64 percent water. By this method any surplus overflow of asphalt from the joints during the filling process may be scraped and peeled off the surface after it has cooled and hardened, leaving a clean surface with flush-filled joints. Lime whitewash may also be used for this purpose. It is prepared by mixing finishing lime and water in the proportions of not less than 1½ sacks of lime to 50 gallons of water. The mixture should be prepared one day in advance of using. Methods for applying these separating agents are described in the report for the United States on brick pavements.

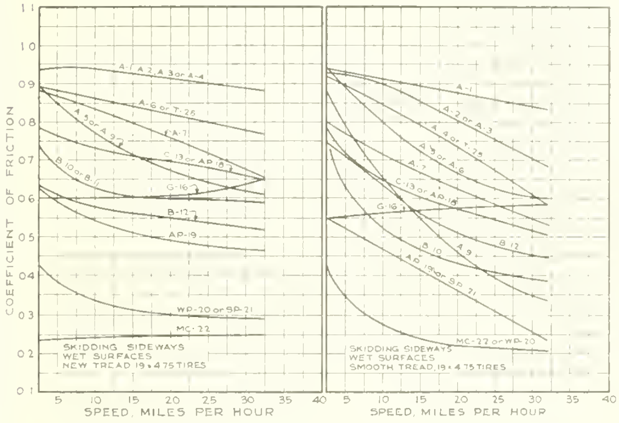
The coefficient of friction of dry brick pavements ranges from 0.7 to 1.0 for straight skidding and from 0.8 to 0.9 for sidewise skidding; the coefficient of friction of wet brick pavements ranges from 0.3 to 0.8 for straight skidding and from 0.4 to 0.7 for sidewise skidding; the ranges in each case covering the use of both new and smooth tires.

4. *Bridge floors.*—Wood plank, smooth steel traffic plates, and ordinary asphalt plank were found in the Moyer tests to be as slippery when wet as icy surfaces. It is particularly hazardous to use these materials on bridge floors. Skidding accidents under such circumstances are more hazardous because of the restricted roadway. Indeed, it is the reporter's opinion that they should not be used under any conditions as a highway surface unless they are provided with some type of nonskid finish. Wood planks may be covered with a bituminous treatment involving the use of cover stone of suitable size so as to give a gritty surface finish. Steel plates should have a stud or grid design providing sharp edges to grip the tires and prevent skidding. Likewise, the surfaces of ordinary asphalt plank should be covered with sharp, gritty aggregate. In general, bridge floors are now covered either with portland cement concrete or high-type asphaltic concrete, both of which afford a satisfactory skid-resistant surface in wet weather.

5. *Sand, gravel, cinder, and macadam surfaces.*—Sand, gravel, cinder, and macadam surfaces all offer a higher resistance to skidding in either the wet or dry condition when the surface is firm and reasonably free from loose material. Loose aggregate, especially sand and gravel pebbles, acts in the same manner as ball bearings under the tires. It causes variations in the skidding resistance, making steering difficult, and in this way causes many accidents. Such surfaces also tend to become dusty in summer, materially reducing visibility, and thus, in another way, creating a hazard. The loose, dusty condition also increases the cost of transportation by causing increased fuel consumption, tire and car repair costs, and road maintenance costs. When there is considerable traffic, the cost of stabilizing the surfaces is unquestionably justified. In parts of the United States clay binder has been used with some success, but it has been clearly demonstrated that the presence of clay in a gravel or macadam surface lowers the chance of success of any subsequent treatment with oil or tar. In the opinion of the reporter, it is much more effective and cheaper in the long run to stabilize such surfaces with an asphalt or tar binder. This can be done either by the road-mix method or by the placing of a mat of oil, asphalt, or tar and screenings, as a wearing course.



COEFFICIENTS OF FRICTION OF TIRES SKIDDING STRAIGHT AHEAD ON WET SURFACES AT SPEEDS RANGING FROM 3 TO 40 M P H (See Figure 1 for key to surfaces



COEFFICIENTS OF FRICTION OF TIRES SKIDDING SIDEWAYS ON WET SURFACES AT SPEEDS RANGING FROM 3 TO 30 M P H (See Figure 1 for key to surfaces)

FIGURE 2.—COEFFICIENTS OF FRICTION OF TIRES SKIDDING STRAIGHT AHEAD AND SIDEWAYS ON VARIOUS TYPES OF ROAD SURFACES.

6. *Maintenance.*—The results of the Moyer tests on mud-, snow-, and ice-covered surfaces, regardless of their construction, clearly indicated that these conditions are the most hazardous the motorist is likely to encounter. Ice and sleet were found to be particularly slippery, providing a frictional resistance of only about one-fifth of that provided by a dry concrete surface. The seriousness of the snow and ice problem as related to skidding is indicated by the fact that about 80 percent of all skidding accidents in Iowa, for example, were found to occur in the 4 winter months.

The correction of this slippery condition is clearly a maintenance problem and demands prompt attention if it is to be effective. Ice forms on the road surface at a temperature close to the freezing temperature, and if sand and cinders are applied promptly at that time, the sand and cinders will be embedded in the ice, thus raising its frictional resistance to 3 or 4 times that which it can provide without such treatment. If the application of cinders or sand is postponed until lower temperatures prevail, the sand or cinders may not be embedded in the ice and the wind and traffic may whip them off the road. It is possible to apply the sand or cinders at lower temperatures if they are first heated or premixed with calcium chloride or rock salt. Such treatment causes the sand or cinders to become embedded in the ice the same as if placed at higher temperatures.

In certain States of the United States, notably Michigan, calcium chloride and sodium chloride have been applied directly on ice-covered surfaces. It has been found that such treatment, when applied to ice overlying portland cement concrete surfaces, causes serious damage to pavement and results in a form of scaling and a gradual disintegration of the concrete. For this reason it is suggested that the use of sodium or calcium chloride be avoided and that the sand or cinders be heated before application to ice-covered portland cement concrete surfaces.

Snow (particularly dry snow) has from two to three times the skid-resistance of ice. However, snow may so suddenly change to ice and is so difficult to distinguish from ice that the driver cannot be sure that a snowy surface is free from ice. It is therefore necessary to apply sand and cinders at intersections, on hills, and on curves as soon as the snow becomes packed. The practice of beginning snow removal coincidently with the beginning of the storm and of carrying on the snow removal operations continuously until the storm has ceased has become a standard procedure with State highway departments in the United States.

Mud on pavements constitutes a serious skidding menace. Where possible, the muddying of surfacings should be prevented by placing gravel coatings on shoulders and on muddy side roads, farm drives, or field entrances.

Roads that have glazed surfaces and are known to be slippery when wet should be covered with nonskid treatments of asphalt and mineral aggregate. Attempts have been made to apply heated sand or screenings on glazed bituminous surfaces, but the results have been more or less ineffectual. Usually the rolling operation cracks the stones, with the result that they are soon loosened by the traffic. Under particular conditions, as where soft asphalt with a low melting point has been used, it is sometimes possible to perform this work satisfactorily but, in general, the fact that the treatment can be given on only a few days of the year and a few hours in each day renders the method highly impractical. Likewise the correction of glazed surfaces by cutting back the excess asphalt with volatile solvents and applying gritty cover stone to blot up the excess binder has met with only mediocre success. The nonskid treatment of pavements by the placing of a mat of asphalt and stone chips has been found to be generally satisfactory, but the successful application of this treatment requires long experience and painstaking workmanship. The methods found most satisfactory in Oregon are described elsewhere in this report. The application of nonskid treatment on old concrete pavement is an even more difficult procedure because of the difficulty of successfully binding thin mats to concrete surfaces.

OBSERVATIONS BY STATE HIGHWAY DEPARTMENTS

Wyoming.—The Wyoming Highway Department reports that practically all of its hard-surface roads are of bituminous type built at rather low cost according to a road-mix specification. The Wyoming experience indicates that when slow-curing oils are used there is danger of their softening during hot weather, allowing the aggregate to settle below the surface and leaving a film of asphaltic cement on the top of the wearing surface. The tendency in their present practice is toward the use of harder asphalt and sealing with a rapid-curing cut-back covered with sharp, clean sand.

Virginia.—The Virginia Department of Highways reports a method of making pavements nonskid that it has used recently on a section of U. S. Route No. 1 between Alexandria and Fredericksburg carrying a traffic of well over 6,000 vehicles daily. An original 18-foot concrete pavement had been widened on each side to provide for 3 lanes, and sheet asphalt had been laid over the entire surface. The sheet asphalt proved slippery during periods of heavy fog and rain, and to eliminate the hazardous condition, a nonskid bituminous surface treatment was applied on the grades.

The treatment consisted of an application of one-quarter gallon of cut-back asphalt heated to a temperature of 100° F. This binder was made from an asphalt having a penetration of 85–100 mixed with naphtha in the ratio of 2 parts asphalt to 1 part naphtha. Before the application of the cut-back binder, all cracks in the pavement were patched by hand. A slag covering was immediately applied at the rate of 20 to 22 pounds per square yard. Extreme care was used to effect a uniform distribution and no brooming was permitted. The slag was rolled lightly with a 5-ton roller, and traffic was kept off the road for a period of about 72 hours. The Virginia authorities report that excellent results were obtained on this section which has now been subject to traffic for about 4 months. They are of the opinion that the closing of the section to traffic for a time sufficient to allow the volatile materials in the asphalt binder to escape was one of the major causes of the successful results obtained.

Texas.—The State Highway Department of Texas reports that the quality of a road surface from the viewpoint of slipperiness or roughness and its resistance to skidding has been the subject of considerable study and research by that department for many years. The early concrete pavements built in the State were finished very smooth and they were found to be quite slippery when wet or damp. The condition has been corrected by the use of a belt on the freshly placed concrete at approximately the time it reaches initial set. It has been found that this finishing treatment raises the grains of sand in the mortar and creates light ridges in the surface which afforded a very satisfactory resistance to skidding. The skill of the workmen in the manipulation of the belt determines the type of surface secured; hence this practice has been subjected to careful control by the Texas Department for the past 7 years.

For the "nonskidding" or roughening of asphalt-surfaced roads the Texas department uses both a penetration and a premix type of treatment. In the penetration type treatment the nonskid surface is provided through the use of hard, angular, crushed stone or gravel placed uniformly over the surface as a wearing course. The amount of asphalt used is carefully controlled as are also the proportions and sizes of the lower course of rock. The entire course must be stable, and the amount of asphalt must be less than that which, penetrating upward through the surface course, would fill the voids between the rock and create a smooth surface. When the amount of asphalt is thus properly controlled, a surface affording satisfactory resistance to skidding is provided.

The premix asphaltic treatments are of two general classifications—hot mix and cold mix. In the cold-mix construction the most recent procedure is to place a light surface course of the open-mix type as the last course of the surface construction. This course is not more than ½-inch thick. The aggregate used is ⅜ inch in size and is very hard and sharp. The mix is so designed that the amount of mortar—that is, the portion of the mix below the 10-mesh screen—is not sufficient to fill the voids in the coarser aggregate and, in consequence, the ⅜-inch material is everywhere evident in the wearing course and provides a considerable resistance to skidding.

In the hot-mix construction, the surface course is placed by means of a mechanical finishing machine to a depth of from ⅜ inch to ½ inch. The aggregate is very hard, ⅜ inch maximum size, and the voids are only half-filled with mortar. The mix is placed at a temperature of 375° F. which creates a segregation due to the latent heat in the coarse aggregate. In normal asphaltic concrete construction, segregation is one of the things that is carefully avoided. However, in placing this nonskid surface it was accidentally found that by creating a segregation the type of surface desired was obtained.

The placing of the mix under the mechanical finishing machine in such a thin layer makes it necessary that it be sufficiently warm to work properly. It was found that when the temperature was raised slightly above the normal temperature—in other words to 375° F.—the manipulation of the surface by the finishing machine and the heat in the coarse rock caused the mortar to slip down into the bottom of the course by gravity which left the top open and the lower half of the large rock firmly embedded in the matrix. This provided the most satisfactory nonskid surface that has yet been laid in the State of Texas. It is obvious that the penetration of the asphalt must be low in order that the mix shall be stable. A 55-penetration asphalt has been used and it has been found that the high heat used in mixing and placing decreases this penetration normally to somewhere between 25 and 35.

Arizona.—The Arizona State Highway Department reports that in general its bituminous surfaces are constructed by the road-mix method, using slow-curing road oils and medium-curing cutback asphalt. These surfaces are covered either with a seal coat of 90 to 95 percent road oil and screenings ranging from ½ inch to those passing a 10-mesh sieve, or with 90 to 95 percent road oil premixed with the screenings of the same size. The department reports that it has, in every case, obtained a roughened surface with the requisite nonskid properties, but in some cases the roughened surface has later lost its roughness due to the softening of the surface and gradual embedding of the screenings until a smooth layer of asphalt appeared on the surface. The department is guarding against this condition by a more thorough study of the design of the mix.

Colorado.—The State Highway Department of Colorado reports that in its road-mix construction (bituminous surfaced roads), it guards against the use of too much asphalt. Surfaces are constructed with the minimum amount of asphalt required to supply the essential binder, the main course being followed by a seal coat containing a cover stone which provides a gritty surface resistant to skidding.

Kansas.—The Kansas Highway Commission calls attention to the prevalence of oil drippings along the quarter points of portland cement concrete pavements and to the resultant low resistance to sidewise skidding at those points. The department is attempting to devise ways and means to eliminate this oil streak as a maintenance process. Variations in the skidding characteristics of surfaces of bituminous macadam and other types of bituminous pavement have caused some concern in this State. To correct such conditions, use has been made of light applications of rapid-curing cutback covered with aggregate graded in size between the ¾-inch ring and the No. 8 square-mesh sieve. The Kansas engineers recommend the use of transition curves for highway alinement which will lessen the throw of the car caused by the sudden application of centrifugal force at the point of the curve. On slippery pavements, this throw frequently induces skidding.

Massachusetts.—The Massachusetts Department of Public Works stresses the banking of curves and the lengthening of curve radii as well as the design and construction of nonskid surfaces to prevent the hazard of motor car skidding. It recommends a surface free from depressions and bumps, and providing a maximum amount of angular particles of the mineral aggregate in contact with the tire and particles sufficiently large to resist the force causing the tendency to skid. The Massachusetts portland cement concrete surfaces have been constructed by the well-known method of brooming and afford sufficient angular contact with the tires.

The bituminous macadam surfaces of Massachusetts are known throughout the United States for their excellent quality. Great care is exercised to obtain the proper balance between the amount and kind of bituminous material and the size of the keystone. During the past few years, however, the Massachusetts department has developed a rather coarse, knobby surface texture without any cover of pea or chip stone. The large stones present a rough mosaic, the top of which is free of asphalt. Clean, properly graded broken stone of the resistance to wear and toughness required is used and this, when rolled, affords a firm, interlocking surfacing. Chip stones are used in quantities sufficient only to fill the voids below the top surfaces of the large stones.

The Massachusetts department has been of the opinion that this open, granular type is best from the viewpoint of skid prevention and that such pavements are as durable, if not more so, than the tighter, mat-covered pavements of former years. However, such roads cause an unpleasant rumble in a rapidly moving closed motor vehicle. The department recognizes this disadvantage and plans to return to the former method using a surface of finer texture. It is to be noted that the department's opinion in regard to the skid-resisting properties of the coarse, knobby type of surface does not coincide with the results of the research of Professor Moyer, which leads to the conclusion that all State departments should calibrate their types of pavement by means of equipment similar to that devised by Professor Moyer to the end that they may speak a common language and refer to definite values of the coefficients of straight-ahead and sidewise skidding. It would likewise be desirable if some standard device to measure the nonskid properties of various types of pavements could be universally employed.

Mississippi.—The Mississippi Highway Department reports that it is rolling bituminous-coated chips into both the Topeka and the sheet asphalt types of asphaltic concrete, thereby obtaining a very nonskid pavement surface.

Wisconsin.—The Wisconsin State Highway Commission has had some difficulty in maintaining bituminous surfaces, due to the flushing of the asphalt to the surface. It has remedied this condition by the application of a seal coat of additional asphalt and hard, sharp stone chips. It reports that it has found that ice melts more quickly on bituminous surfaces than on concrete, due presumably to the greater absorption of heat by the darker material. Some difficulty has been experienced in sanding ice covering portland cement concrete pavement because of injurious effects of the salts mixed with the sand and cinders, which salts apparently have caused a later disintegration of the concrete surface.

Oregon.—As has been stated, it is necessary to build and maintain smooth-riding and skid-resisting surfaces. Rough pavements are the direct cause of skidding at high speed. The State Highway Commission of Oregon has devoted a great deal of time and study to devising ways and means for building smooth pavements of various types. It is axiomatic, however, that no matter how smooth pavements may be when originally constructed, they eventually become rough. Grade settlements, slight land movements, occasional weakness in foundations, and faults in drainage all tend to deform the pavement. Sooner or later, therefore, it becomes necessary to patch all pavements. Hence, it would seem to be highly desirable, if not necessary, to develop a technique and workmanship resulting in smooth patches. If patches are smooth, it should be possible to keep pavements in good condition, if not indefinitely, at least over long periods of time.

The Oregon department patches all pavements, regardless of type, with hot asphaltic concrete. The patches are nonskid in character and feathered to paper thinness at the edges. Dark asphaltic concrete patches on portland cement concrete pavements are objectionable to some extent, but they are few in number during the early life of the pavement and disfigure it less than might be supposed.

Portland cement concrete pavements are made smooth at the time of their construction by means of delayed finishing; that is, by delaying a part of the finishing until the shrinkage incident to the initial setting of the concrete has taken place, and by the further procedure of scraping down the high spots with heavy steel scrapers after the initial set has occurred. Very little scraping work is actually performed, however, as contractors, in order to avoid the expense of the scraping, perform the preceding finishing operations with the utmost care. Scraping is objectionable in that it removes the broom finish, leaving the scraped area smooth and less nonskid; but, as stated, the total area scraped is very small.

After extensive trial, the Oregon department has abandoned the mudjack as a means of pavement repair, having found that its use is only partially satisfactory in restoring the smoothness of the surface and that it results in much breakage of slabs into small sections which tend to rock and move under traffic. As the portland cement concrete pavements become progressively rougher, the asphaltic concrete patches increase in number until finally they constitute a complete resurfacing, with the concrete pavement serving as a base for an asphaltic concrete top.

Bituminous pavements constructed in Oregon are of two types, the plant-mix type and the penetration type. All plant-mix construction is of high-type asphaltic concrete of which there are two general classes, the minimum-void or closed-type mixes, and the open mixes. It is the theory of the reporter that it is very difficult to maintain a permanently nonskid surface with the densely graded mixtures for the reason that there are practically no voids for the asphalt to occupy under the influence of heat and the constant pounding of traffic. There is, therefore, the danger of free asphalt exuding upon the surface. It is possible, but rather difficult, to avoid this by a rigid control of the proportions of the aggregate and the percentage of asphalt, using the minimum of asphalt requisite to keep the mixture sealed and to prevent raveling in cold or wet weather. Very good results have been obtained in Oregon by exercising this rigid control, and pavements 5 years of age have retained the nonskid features required for safe driving.

The seal coat is eliminated from this specification and ½-inch screenings coated with 2 percent asphalt cement are rolled into the surface mixture, while it is still warm, at the rate of approximately 5 pounds per square yard. The proportions of aggregate and asphalt binder illustrating this type of construction are as follows:

	Percent
Aggregate:	
Passing 1¼-inch screen	100
Passing 1¼-inch and retained on ¾-inch screen	16–24
Passing ¾-inch and retained on ¼-inch screen	24–36
Passing ¼-inch screen and retained on 10-mesh sieve	12–20
Passing 10-mesh and retained on 200-mesh sieve	20–28
Passing 200-mesh sieve	3–6
Asphalt, 50–60 penetration	4. 5–6. 5

The open-type mix has sufficient voids for any probable expansion of the asphalt. When properly constructed, no evidence of the flushing of the asphalt to the surface has been noted. This construction is used as a wearing course only, and is placed upon an impervious, closed mix as a base. The grading of the materials and the percentage of asphalt are as follows:

	Percent
Aggregate:	
Passing ¾-inch screen	100
Passing ¾-inch and retained on ¼-inch screen	45–55
Passing ¼-inch screen and retained on 10-mesh sieve	30–36
Passing 10-mesh and retained on 200-mesh sieve	8–14
Passing 200-mesh sieve	0–2
Cut-back asphalt (RC–4)	4–6

In closed mixes, an asphaltic cement of 50–60 penetration is employed. In open mix, a heavy-grade cut-back asphalt known as RC–4 is used, in which asphalt of 80–120 penetration is cut back with a maximum of 15 percent of naphtha as determined by distillation at 680° F. All types of asphaltic concrete mixtures are laid with finishing machines. The dense mixtures are laid at a temperature of 325° F. and the open type mixtures at temperatures ranging from 175 to 250° F.

In the early practice of the Oregon State Highway Commission, some rather high-crowned asphaltic concrete pavements, 16 feet in width, were constructed. These pavements were covered with a heavy seal coat of asphaltic cement and sand and they became very slippery in wet weather. It therefore became necessary to eliminate this very definite hazard. The first treatments consisted of one application of asphaltic cement or tar followed by a covering of stone chips or pea gravel. The results were, on the whole, mediocre. The stone tended to whip off the high spots, producing a surface of varying texture. Certain areas were covered with stone chips or pea gravel, affording a nonskid texture, while other areas were entirely bare of screenings and therefore slippery. It was finally found necessary to plane off the excess of asphalt from the surface of the old pavement. The asphalt and fine mineral material had accumulated in bumps and in certain cases had formed rhythmic corrugations.

The planing machine consisted of a heavy trailer of long wheelbase, carrying adjustable chisel-like cutting edges, pulled by a 60-horsepower crawler-tread tractor. The chisels were kept sharp and not only cut off the asphalt-sand mastic, but also sheared through the stones in the underlying mix. Repeated trips with the planer not only removed the excess of asphalt but also produced a very smooth-riding surface. The removal of the excess of asphalt eliminated any chance of its being drawn up into the new wearing course. Upon the planed surface was constructed, by the penetration method, a wearing course approximately 1¼ inches in thickness. Stone of maximum 1¼-inch size was used. The asphalt content was held to the minimum and excellent results were obtained. Surfaces treated in this manner some 8 years ago are practically as skid-resistant as when first constructed.

Probably the most uniform nonskid texture pavement laid in the State of Oregon is the penetration macadam type. The base stone, of 2½-inch to 1¼-inch size, is penetrated by hot asphaltic cement and covered with stone ranging from 1¼ inches to ¾ inch in size. This stone is carefully leveled by long-wheelbase motor graders. After the second application of asphaltic cement, the bituminous macadam is keyed with ¾- to ½-inch stone. This is followed by the first seal coat using ½ to ¼-inch and ¼-inch minus cover stone. Traffic is turned onto the pavement 4 or 5 days before the application of the second seal coat, after which the surface is thoroughly swept and the second seal employing the same size of aggregate is placed. This method effectively seals the penetration macadam against the admission of moisture. Hot asphaltic cement of 120–150 penetration, heated to a temperature of 425° F., is used in all cases with the exception of the last seal coat. A water-soap solution, in the amount of 10 percent of the hot asphalt by volume, is added through separate openings in the spray nozzles. The temporary emulsion formed covers the stone thoroughly. Either cut-back asphalt or emulsified asphalt is used for the second seal coat.

The surface texture of this type of pavement is somewhat coarser than a sandpaper finish, the ½-inch stone and some of the ¼-inch stone being present in the surface. The voids in the base stone act as a reservoir to absorb excessive asphalt in hot weather with the result that this type of pavement seldom bleeds and remains remarkably uniform in texture throughout the years. Oregon has had about 10 years' experience with this type of surfacing. In all, 2 gallons per square yard of bituminous cement, including the cut-back or emulsion, are required. The total aggregate used amounts to 0.111 cubic yards per square yard. Each course of the pavement is thoroughly rolled with 10-ton, 3-wheeled rollers.

B. PROPERTY OF REFLECTING OR ABSORBING LIGHT (UNDER ARTIFICIAL LIGHTING)

In view of the fact that consideration is now being given to the artificial lighting of certain sections of rural highways in the United States to eliminate the hazards of night driving, information concerning the quality of road surfaces in respect to their ability to reflect or absorb light is important. Moreover, since the majority of rural highways will, in all probability, remain unlighted on account of the expense, it is of prime importance that those properties of road surfaces which minimize the glare of approaching headlights, particularly on rainy nights, shall be carefully studied.

The reporter has endeavored unsuccessfully to obtain through published articles data relating to visibility and glare in connection with the various types of pavement surface. Apparently few data of this kind have been assembled in the United States up to the present time. Accordingly a series of experiments of limited scope was undertaken by G. S. Paxson, Bridge Engineer, and J. D. Everson, Designer, both of the Oregon State Highway Department. The character of the equipment and the time available prevented any exhaustive study, but the results obtained give at least qualitatively a comparison between the various pavement surfaces examined.

DESCRIPTION OF INVESTIGATION

Investigations were made to determine the light-reflecting properties of various types of pavements under artificial lighting conditions and to measure the reflection or glare from approaching headlights under both wet and dry conditions. Investigations to determine light-reflecting properties under artificial lighting conditions were made on 7 different types of pavement as follows:

1. A new concrete pavement completed and cured but not yet opened to travel. This pavement is broom-finished. The grooves are between 1/16 and ⅛ inch in depth, running transversely to the pavement. Duplicate tests were made with the incident light across the grooves as would be the case in normal lighting and with the incident light parallel to the grooves simulating a pavement broom-finished parallel to the highway center line. These two conditions are referred to in the accompanying charts as "new concrete parallel with grooves" and "new concrete across grooves," respectively.

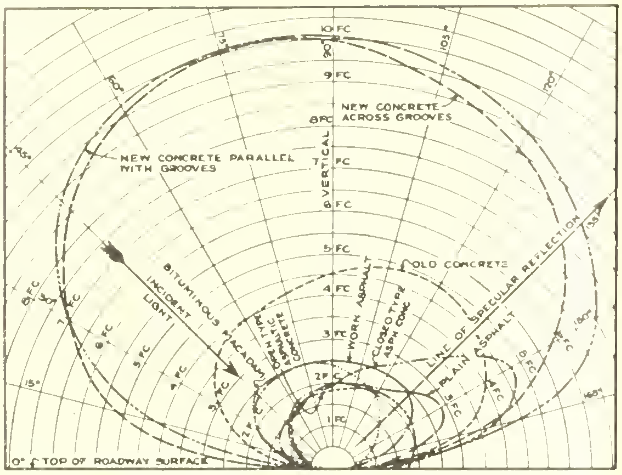
2. A concrete pavement that has been in use 18 years. This pavement was constructed with a smooth surface and has been considerably stained and discolored by oil during service. The pavement is called "old concrete" in the accompanying figures.

3. A section of old asphaltic concrete pavement quite rich in asphalt and showing no aggregate on the surface. This pavement is called "plain asphalt" in the accompanying figures.

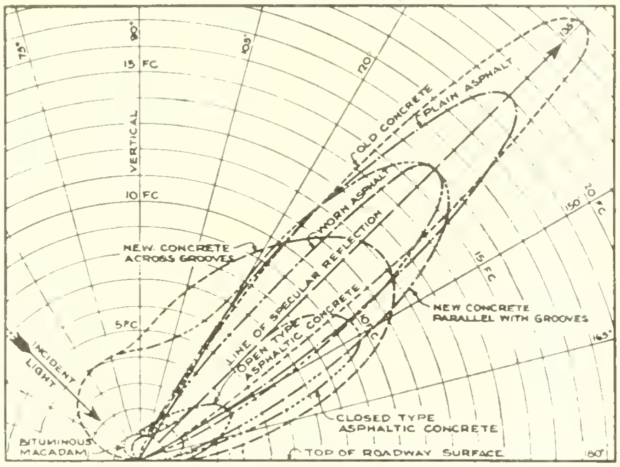
4. An asphaltic concrete pavement that has been in service 15 years. The surface is quite smooth, but over at least 30 percent of the surface small aggregate particles protrude above the shiny asphalt finish. This pavement is called "worn asphalt" in the accompanying figures.

5. A modern asphaltic concrete wearing surface of the "closed type" as described in the first section of this report. This pavement is 3 years old and is called "closed-type asphaltic concrete" in the accompanying figures.

6. An asphaltic concrete pavement of the "open type" as described in the first section of this report. This



LIGHT SOURCE AT 45° TO PAVEMENT SURFACE
REFLECTION IN PLANE OF INCIDENT LIGHT
DRY PAVEMENT SURFACES



LIGHT SOURCE AT 45° TO PAVEMENT SURFACE
REFLECTION IN PLANE OF INCIDENT LIGHT
WET PAVEMENT SURFACES

Figure 3.—Luminosity of Various Types of Road Surfaces When Dry and When Wet, for Light Source at 45° to Pavement Surface.

pavement is 1 year old and is called "open-type asphaltic concrete" in the accompanying figures.

7. A section of bituminous macadam in service about 1 year. The surface of this pavement is composed of stone chips from ¾ to ½ inch in size which project above the body of the pavement about ⅛ inch. This pavement is called "bituminous macadam" in the accompanying figures.

In fixed highway lighting the fixtures are suspended above the roadway surface at a considerable height, and the angle at which the light strikes the surface is controlled by the fixture. With a mounting height of 30 feet and 200 feet between lights, the flattest angle will be about 17°, assuming that each light illuminates the surface halfway to the adjoining light. To simulate these conditions, measurements of the reflected light were made with an incident angle of 45° and with an incident angle of 17°. The light source was a 6-volt incandescent light mounted upon a tripod so that the angle between the light beam and the pavement surface could be accurately determined. The intensity of the light was controlled by a rheostat and voltmeter so that it was kept constant during the entire series of tests.

Reflection readings from the pavement surface, measuring the relative intensity of the light, were taken with a photometer attached to an arm which kept it at a constant distance from the point of reflection on the pavement surface. The intensity was measured in foot-candles. The angle which the axis of the photometer made with the pavement was measured on a dial attached to the frame supporting the photometer arm. The angle of incident light was kept constant and the intensity of the reflection was measured at each 15° around the semicircular arc. Readings were taken in the plane of the incident light.

Readings were first taken on the pavement surfaces when dry. The surfaces were then artificially wetted to simulate conditions during a rain, and the readings repeated.

RESULTS OF THE INVESTIGATIONS

The results obtained with the light at an angle of 45° are shown in figure 3. Those above are for dry surfaces and those below are for the same surfaces when wet. It is interesting to note the regularity of the curves for dry pavement which show almost complete absence of specular reflection or glare except for the three smooth pavements investigated. These were the old concrete, worn asphalt, and plain asphalt. In addition, the closed-type asphaltic concrete, which is a relatively nonskid pavement, shows a decided bulge in the curve of readings taken for dry pavement. The curves for these pavements show a noticeable bulge in the direction of the line of specular reflection.

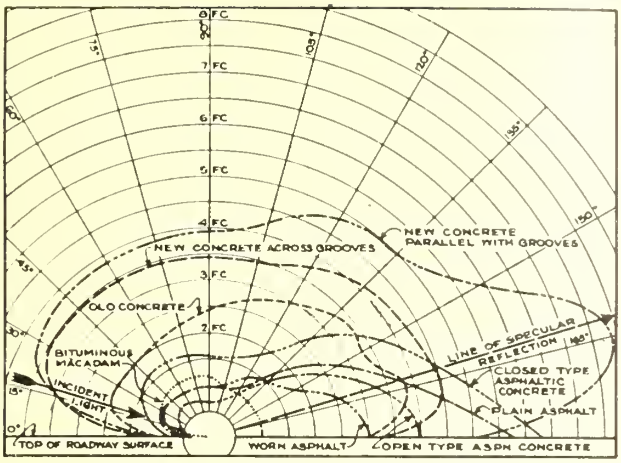
The difference between the luminosity of the dry portland cement concrete pavement and the dry asphaltic concrete pavement is quite noticeable. The new portland cement concrete pavements show about 4 times the luminosity of the asphaltic concrete pavements, while the old concrete pavements show about twice the luminosity. The rough and irregular bituminous macadam surface shows the property of being more highly luminous in the direction of the incident light than it is away from it.

These surfaces were then wetted and tested, other conditions remaining constant. The characteristics of the pavement have entirely changed and a great amount of specular reflection or glare is apparent. It will be noted that the new smooth pavements give from 4 to 10 times the intensity of glare produced by the rough-textured pavements.

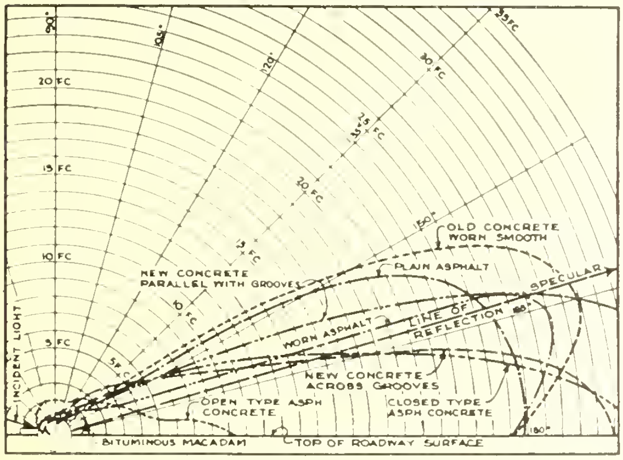
The comparison of the two sets of curves shows that the luminosity of the pavements, except on the line of specular reflection, is one-half or less when wet than when dry; moreover, that in the case of plain asphalt or worn asphalt pavements there is practically no luminosity outside of the glare. This is supported by driving experience which discloses that the pavement appears very bright directly in the line of light sources but practically black at any other angle. It will be noted that while the luminosity of dry bituminous macadam pavement is only about one-fourth that of the new concrete, the glare, when wet, is less than one-sixth as great.

Figure 4 shows the reflection readings on dry and wet pavements with the angle of incident light at 17° with the pavement surface and with the readings taken in the plane of incident light. It should be noted that as the angle of incident light decreases, the glare increases with all types of pavement, wet or dry.

As in the preceding figures, the luminosity of the dry portland cement concrete pavement is from 2 to 4 times that of dry asphaltic concrete types. The spec-



LIGHT SOURCE AT 17° TO PAVEMENT SURFACE
REFLECTION IN PLANE OF INCIDENT LIGHT
DRY PAVEMENT SURFACE



LIGHT SOURCE AT 17° TO PAVEMENT SURFACE
REFLECTION IN PLANE OF INCIDENT LIGHT
WET PAVEMENT SURFACE

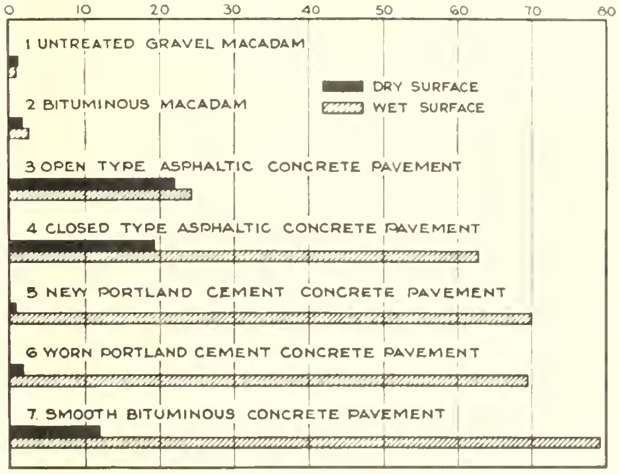
FIGURE 4.—LUMINOSITY OF VARIOUS TYPES OF ROAD SURFACES WHEN DRY AND WHEN WET, FOR LIGHT SOURCE AT 17° TO PAVEMENT SURFACE.

ular reflection or glare from the plain asphalt and worn asphalt types, when dry, is from 2 to 3 times that of the luminosity in other directions. The reflection from concrete pavements, when wet, is 4 to 8 times the luminosity of these same pavements when dry. Bituminous macadam, when dry, shows practically no glare at all.

An examination of the lower part of figure 4 shows that none of the pavements has an appreciable luminosity when wet and that all but the bituminous macadam evidence some glare. An interesting point in this study is the decreased glare from portland cement concrete pavements when the light is across the grooves as compared with the same pavement when the light is parallel to the grooves.

From this study, it appears that under dry conditions, new, clean portland cement concrete pavements are more easily lighted than any of the darker asphaltic types. This is undoubtedly due to the light color of the portland cement concrete pavement. The rough open-textured bituminous macadam surface produces the least glare when wet.

It was noted during the experiment simulating rain that the smooth pavements were covered by a practically continuous sheet of water. Even the new concrete with its shallow broom marks became completely flooded. The high surface tension resulting from the small diameter of the grooves prevented the rapid run-off of the water. This suggests the formation of deeper, wider grooves in concrete pavement to afford quick run-off. The irregularities in the surface of bituminous macadam and new-type asphaltic concrete afford fissures or channels wide and deep enough to minimize the surface tension of the particles of water. Such surfaces are seldom entirely flooded and the stone chips projecting above the water surface break up the reflection and greatly reduce the glare. It is thus apparent that the glare is caused in the main by the reflection of the light from the sheet of water acting as a mirror Means should be provided for constructing all types of pavement so as to afford rapid run-off of surface water. This is easily accomplished by the standard methods of building penetration macadam and the open-textured asphaltic concrete of the present day.



PERCENTAGE OF INTENSITY OF ILLUMINATION
DUE TO SURFACE REFLECTION

FIGURE 5.—PERCENTAGE OF INTENSITY OF ILLUMINATION DUE TO SURFACE REFLECTION FOR VARIOUS TYPES OF ROAD SURFACES.

There is room for improvement in portland cement concrete pavements and it is suggested that consideration be given to the scratching of grooves approximately ¼ inch in depth 1 inch apart, transversely across the pavement surface, while the concrete is still green.

INVESTIGATION OF HEADLIGHT GLARE

In this part of the investigation, a 1937 model automobile with the headlights in the depressed or dim position was used as the light source. In this position the center of the light beam struck the pavement approximately 65 feet from the light. Readings of the intensity of the light at double this distance, or 130 feet, at the same elevation as the headlights, were taken with a photometer. A screen was then placed in the proper position so as to allow only the reflected light to reach the photometer. This eliminated the direct light from the headlights but measured the specular reflection or glare from the pavement surface.

The same pavement types previously described were tested, except that an untreated gravel macadam surface was added and the plain asphalt type was eliminated.

Figure 5 represents graphically the percentage of the total light reaching the photometer which was reflected

(Continued on page 121)

STUDY OF ROAD SUBSOILS

(A) DETERMINATION OF PROPERTIES OF SUBSOILS, TESTING METHODS, EQUIPMENT; (B) INFLUENCE OF THESE PROPERTIES ON ROAD CONSTRUCTION (BASE AND SURFACE) AND ON ROAD MAINTENANCE

By **C. A. HOGENTOGLER**, Senior Highway Engineer, United States Bureau of Public Roads, Principal Reporter

A STUDY of road subsoils is made quite generally by the individual highway departments of the United States in connection with the design, construction, and maintenance of their highway systems. The experience of the United States Bureau of Public Roads since the inception of its soil research program in 1920, supplemented by numerous contributions from other soil investigators in the highway field, has disclosed three major fields in which soil information and studies have distinct economic value. They are: (1) The use of the subgrade survey and the classification of soils into groups based on performance, (2) the construction of "stabilized" road surface and base courses, and (3) the construction of highway embankments.

SUBGRADE SURVEY AND SOIL GROUPING

The subgrade survey furnishes information which assists in locating the road, selecting the most satisfactory type of surface and eliminating conditions likely detrimentally to affect the serviceability of the pavement.

Examinations of the soil in its natural field condition, in excavations or road cuts or, more commonly, by means of a soil auger, are made at intervals close enough to determine the soil type and to depths sufficient to penetrate the more-or-less nonuniform layers of soil or soil material.

Representative samples obtained as part of the subgrade survey are tested in the laboratory to determine their physical properties.

The principal laboratory determinations used in the classification of subgrade soils are the mechanical analysis, and the plasticity, shrinkage, and moisture equivalent tests.

Procedures for these determinations[1] and for the surveying and sampling of soils[2] have been published by the American Association of State Highway Officials.

The groups into which subgrade soils have been tentatively arranged, are described briefly as follows:

Group A-1.—Well-graded material, coarse and fine; excellent binder. Highly stable under wheel loads irrespective of moisture conditions. Excellent surface course; excellent base course, provided plasticity index is not greater than 6; excellent subgrade for pavements of both flexible and rigid types; excellent fill material.

Group A-2.—Coarse and fine materials, improper grading or inferior binder. Highly stable when damp. Friable varieties become loose and dusty in long-continued dry weather. Plastic varieties likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer. Satisfactory subgrade for both flexible and rigid pavements under well-drained conditions. Satisfactory fill material.

Group A-3.—Coarse material only, no binder. Lacks stability under wheel loads, but is rarely affected by moisture conditions. Not likely to heave because of frost nor to shrink or expand in appreciable amount. Furnishes excellent support for flexible pavements of moderate thickness and for relatively thin rigid pavements. Excellent fill material if protected from erosion.

Group A-4.—Silt soils without coarse materials and with no appreciable amount of sticky colloidal clay. Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated. When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load. May cause cracking in rigid pavements due to frost heave and failure in flexible pavements because of low supporting value. Not satisfactory for surface or base course. Generally satisfactory for subgrade or fill material under controlled conditions.

Group A-5.—Similar to Group A-4, but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry. Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards.

Group A-6.—Clay soils without coarse material. In stiff or soft plastic state absorb water only if manipulated. May then change to liquid state and work up into the interstices of macadams or cause failure due to sliding in high fills. Furnish firm support essential in properly compacting macadams only at stiff consistency. Deformations occur slowly and removal of load causes very little rebound. Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking in rigid pavements.

Group A-7.—Similar to Group A-6, but at certain moisture contents deforms quickly under load and rebounds appreciably on removal of loads, as do subgrades of Group A-5. Alternate wetting and drying under field conditions leads to even more detrimental volume changes than in Group A-6 subgrades. May cause concrete pavements to crack before setting and to crack and fault afterwards.

Group A-8.—Very soft peat and muck incapable of supporting a road surface without being previously compacted.

The test data limits which identify soils of the different groups have been discussed elsewhere.[3]

Subdivisions of Groups A-5, A-6, A-7, and A-8.—All varieties of Groups A-5, A-6, A-7, and A-8 are unsuitable for surface or base courses. Soils of these groups having liquid limits of 45 or less are suitable for subgrade and fill material under controlled conditions. Soils of these groups having liquid limits greater than 45, but not greater than 65, are poor materials for subgrades or fills and should be used only under rigidly controlled conditions of moisture content and compaction. Soils of these groups having liquid limits greater than 65 are unsuitable for subgrade and fill construction.

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[1] Methods T-87, T-88, T-89, T-90, T-91, T-92, T-93, and T-99. Standard Specifications for Highway Materials and Methods of Sampling and Testing. American Association of State Highway Officials. Washington, D. C., 1935.

[2] Standard Method of Surveying and Sampling Soils for Use in Place as Subgrades for Highways. Method T-86, Standard Specifications for Highway Materials and Methods of Sampling and Testing. American Association of State Highway Officials, Washington, D. C., 1935.

[3] Subgrade Soil Constants, their Significance and their Application in Practice, part II. A discussion of the Soil Constants and the Soil Identification Chart. C. A. Hogentogler, A. M. Wintermyer, E. A. Willis, PUBLIC ROADS, vol. 12, No. 4, June 1931.

STABILIZED ROAD SURFACE AND BASE COURSES

Thousands of miles of "stabilized" road surface and base courses have been built in the United States with results which justify the continuance of this type of construction.

The methods of soil stabilization utilized depend on the materials locally available. In some locations, such as the southeastern part of the United States, nature provided deposits of sand-clay and top soil having the grading and character required in the best of soil roads or base courses.

In other locations, binder soils and aggregates are available for producing mixtures having the properties of the best naturally good soils. At times materials are available for the gradations required in roads, but the binder soil may be of inferior quality. Finally, there are locations where there is a deficiency of the aggregate required for properly graded mixtures.

Studies of the best natural road soils have indicated that the design requirements of stable soil mixtures should include the following:

1. The aggregate should be hard and durable enough to resist weathering, traffic abrasion, and crushing. Sound, tough particles or fragments of gravel, stone, slag, or combinations of them, crushed to the proper size, should prove suitable. Certain types of shales and similar materials that break and weather rapidly when alternately frozen and thawed, or wetted and dried, should not be used.

2. The binder material of the soil, known as fines, should be of a character such as to provide graded mixtures with the proper balance of capillarity and adhesion without risk of detrimental volume change. It is particularly essential that the fines do not swell enough in the presence of moisture to cause the clay to become a lubricant instead of a binder.

3. When local materials are available for the proper proportioning of aggregate and binder soil, but the natural clay does not have the binder value required in highly stable road surfaces, a number of admixtures may be used singly or in some combination. There are, first, the chemical salts, calcium chloride, sodium chloride, and magnesium chloride; and second, the waste products of industry, such as sulfite liquor from the manufacture of wood pulp, sucrate liquor from the molasses industry, and huge accumulations of waste sizes of the mineral aggregate industry.

4. When only fine or poorly graded materials are available, the asphaltic, tar, and portland cement binders may be utilized to provide stable base courses to be covered with bituminous surfacings. In isolated cases, clay soils have been heated to high temperatures by specially constructed furnaces which are moved slowly over the locations to be improved.

GRADED SOIL MIXTURES

The suitability of graded mixtures has been determined by means of sieve analysis of the aggregate retained on the No. 10 sieve, combined sieve and hydrometer analysis (Method T–88, A. A. S. H. O.) of the fraction passing the No. 10 sieve, and the plasticity tests (Methods T–89, T–90, T–91, A. A. S. H. O.) performed on the fraction of the material passing the No. 10 sieve, an effort being made to combine materials in such quantities that the resulting mixture will conform to the requirements of stabilized road surfaces or bases.

Specifications proposed to the American Association of State Highway Officials for both surface and base course materials contain essential requirements as follows:

1. Surface course.

Type A.—Sand-clay mortar.

Passing—	*Percentage by weight*
1-inch sieve	100
No. 10 sieve	65–100

The material passing the No. 10 sieve should have gradings as follows:

Passing—	*Percentage by weight*
No. 10 sieve	100
No. 20 sieve	55–90
No. 40 sieve	35–70
No. 200 sieve	8–25

Type B.—Coarse graded aggregate.

Passing—	*Percentage by weight*
1-inch sieve	100
¾-inch sieve	85–100
⅜-inch sieve	65–100
No. 4 sieve	55–85
No. 10 sieve	40–70
No. 40 sieve	25–45
No. 200 sieve	10–25

Type C.—Crusher run materials.

Passing—	*Percentage by weight*
¾-inch sieve	100
No. 4 sieve	70–100
No. 10 sieve	35–80
No. 40 sieve	25–50
No. 200 sieve	8–25

The fractions of surface course materials, A, B, and C, passing the No. 200 sieve should be less than ⅔ of the fraction passing the No. 40 sieve. The fraction passing the No. 40 sieve should have a liquid limit not greater than 35 and a plasticity index not less than 4 nor more than 9.

2. Base course.

Type A.—Sand-clay.

Passing—	*Percentage by weight*
No. 10 sieve	100
No. 20 sieve	55–90
No. 40 sieve	35–70
No. 200 sieve	8–25

Types B–1 and B–2.—Coarse graded aggregate. (See table 1.)

TABLE 1.—*Grading of base course aggregates, types B–1 and B–2*

Passing—	B–1 1-inch maximum size	B–2 2-inch maximum size
	Percent [1]	*Percent* [1]
2-inch sieve		100
1½-inch sieve		70–100
1-inch sieve	100	55–85
¾-inch sieve	70–100	50–80
⅜-inch sieve	50–80	40–70
No. 4 sieve	35–65	30–60
No. 10 sieve	25–50	20–50
No. 40 sieve	15–30	10–30
No. 200 sieve	5–15	5–15

[1] By weight.

The fraction of base course materials, A, B, and C, passing the No. 200 mesh sieve should be less than ½ of the fraction passing the No. 40 sieve. The fraction passing the No. 40 sieve should have a liquid limit not greater than 25 and a plasticity index not greater than 6.

Type C—Crusher-run materials.

Passing—	*Percentage by weight*
¾-inch sieve	100
No. 4 sieve	70–100
No. 10 sieve	35– 80
No. 40 sieve	25– 50
No. 200 sieve	8– 25

STANDARD METHOD OF TEST FOR DENSITY OF SOIL

In stabilizing fine grained soils with an adhesive binder such as portland cement or bituminous material, it is necessary to compact them to high density. This is best accomplished at a definite moisture content, known as the optimum, for each mixture. The test used to determine the optimum moisture content is known as the density test[4] and is described as follows:

1. *Apparatus.*—The apparatus shall consist of the following: A cylindrical metal mold, known as the density or Proctor cylinder, approximately 4 inches in diameter and 4½ inches in height and having a capacity of 1/30 cubic foot. This mold is fitted with a detachable base plate and a removable extension approximately 2½ inches in height.

A metal tamper having a striking face 2 inches in diameter and weighing 5½ pounds.

A steel straightedge 12 inches long.

A scale of 25-pound capacity sensitive to 1/100 of a pound.

A 100-gram capacity balance sensitive to 1/10 gram.

A suitable drying oven.

Miscellaneous equipment including porcelain dishes for moisture determinations, beakers, etc.

2. *Sample.*—A 6-pound sample, air dried to slightly damp condition, shall be taken from a portion of the material passing the No. 4 sieve.

3. *Procedure.*—The sample is thoroughly mixed and compacted in the cylinder in 3 equal layers, each layer receiving 25 blows from the tamper which is dropped from a height of 1 foot above the soil. The soil is then struck off to the level of the cylinder and the weight determined. A small sample of the compacted soil is oven dried to determine the moisture content. The soil is removed from the cylinder and broken up until it will pass a No. 4 sieve, water is added, and the above procedure is repeated. This series of determinations is continued until the soil becomes very wet and there is a substantial decrease in the wet weight of the compacted soil.

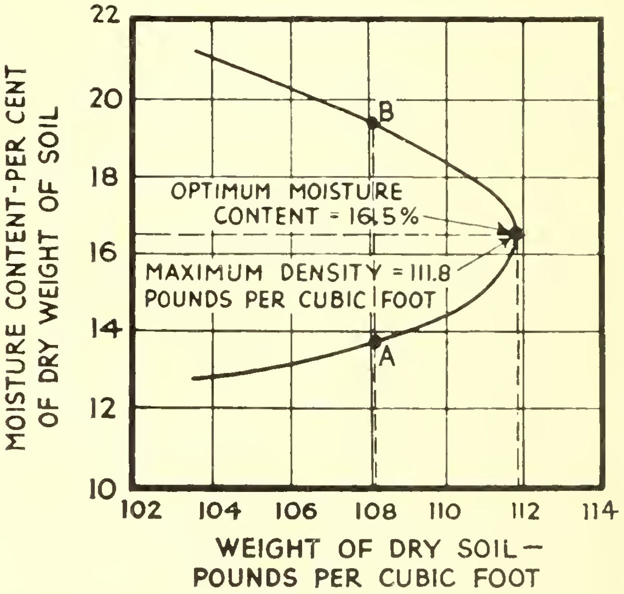
4. *Calculations.*—The moistures shall be computed by means of the following formula:

$$\text{Percent moisture} = \frac{\text{weight of water}}{\text{weight of oven dried soil}} \times 100$$

The dry weight shall be computed by means of the following formula:

$$\text{Dry weight} = \frac{\text{wet weight in pounds per cubic foot}}{\text{percent of moisture} + 100} \times 100$$

The results of the compaction test, corrected for weight of moisture and expressed as pounds of dry soil per cubic foot, are plotted against their respective moisture contents and a smooth curve drawn through the resulting points. The peak of the curve represents the maximum density for the given material under the above compaction. The percentage of water at which maximum density is attained is termed the optimum moisture content.



Moisture-Density Curve Showing Typical Result of Soil Density Test.

STABILIZATION WITH PORTLAND CEMENT

Soils.—The soils for this work consist of the material in the subgrade within the lines, grades, and cross sections indicated on the plans, except as otherwise designated by the engineer. Soils are sampled and tested in accordance with the requirements of the A. A. S. H. O. methods T–86 to T–94, inclusive. Soils having characteristics which will make pulverization and mixing with cement extremely difficult are removed and replaced with suitable soil obtained from borrow pits shown on the plans.

Design of soil and cement mixture.—The percentage of cement as based on compacted dry soil volume is determined by mixing the soil and cement in several arbitrarily chosen proportions (4.6 and 8 percent cement have been found satisfactory for most soil types).

The optimum moisture content and maximum density for each mixture are found by means of the Standard Method of Test for Density of Soil. When the optimum moisture content for each mixture has been found, 4 specimens of each mixture are prepared in the Proctor cylinder. Two of the specimens are subjected to alternate wetting and drying and 2 to alternate freezing and thawing.

One cycle of wetting and drying consists of 8 hours wetting by immersion at room temperature, 15 hours drying at 300° F. and 1 hour cooling at room temperature.

One cycle of freezing and thawing consists of 8 hours wetting by immersion at room temperature; 16 hours freezing at 20° F. (or lower); 8 hours thawing at room temperature; 15 hours drying at 300° F.; and 1 hour cooling at room temperature.

After each cycle of wetting and drying or freezing and thawing, one specimen is brushed with a wire brush to remove visible loose particles, and the other is left undisturbed.

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[4] Stabilized Soil Roads. C. A. Hogentogler and E. A. Willis, PUBLIC ROADS vol. 17, No. 3, May 1936.

The weight and dimensions of all the specimens are recorded after each cycle. The volume change of each specimen is calculated and recorded. A loss of 50 percent by volume of any specimen is considered as the point of failure. Failure in less than 15 cycles of wetting and drying and less than 10 cycles of freezing and thawing is considered as the standard for the elimination of any soil-cement mixture for use in base course construction.

The moisture-compaction curve for the cement-soil mixture chosen is used in obtaining the required moisture and compaction during construction.

STABILIZATION WITH BITUMINOUS BINDERS

Bituminous materials used in the stabilization of fine-grained or poorly graded soils for use in road bases may be arranged in 3 groups as follows: (1) emulsions, (2) tars, and (3) road oils and cut-backs.

Soil-emulsion mixtures.—Absorption and stability tests have been largely used in determining the proper amount of bituminous emulsion to be used in the stabilization of fine grained soils.[5]

Preparation of samples.—Samples of soil as received from the field are dried, broken up and passed through a ¼-inch screen. The material passing is quartered and plastic limit, liquid limit, and particle size determinations are made. Mixes are made using various arbitrary percentages of emulsion. The soil is mixed with diluted stabilizer, or first with water and then with stabilizer, until the asphalt is uniformly distributed and the mass is at a consistency between the plastic and liquid limits.

The mixed soil is allowed to dry to a stiff molding consistency and is tamped into 2-inch metal molds 4 inches long. After further drying, but while still slightly damp, it is compressed, usually under a load of 1,000 pounds per square inch, and is then pressed out of the mold and placed in a drying oven or cabinet and maintained at about 140°F. until at a constant weight. Mixing may be done by hand or with any suitable equipment and the details of the tamping and compacting methods are unimportant because the ultimate compaction and density are obtained by shrinkage during the drying operation.

Absorption test.—The specimens prepared and dried are then ready for the capillarity absorption test.

The absorption test is made in a closed cabinet containing pans filled first with a layer of standard Ottawa sand and then a layer of diatomaceous earth covered with blotting paper, with water-feeding apparatus which keeps the water level exactly at the under side of the blotting paper.

Although the humidity within the cabinet remains very high due to the continuous evaporation of water, the individual specimens are wrapped in cellophane on the sides and covered on top with foil or shallow aluminum cups slightly larger in diameter than the specimen. This prevents the escape of any water taken up by capillarity, and is believed to approximate the exposure of any 2-inch cylindrical portion of stabilized soil in a base where it is exposed to water at the under side, where evaporation is prevented by the surrounding stabilized soil, and where surface evaporation is restrained by surfacing.

Specimens are usually removed for daily weighings and the period of test may be 7 to 90 days. It will be found that the absorption decreases rapidly after the first day because the capillarity of stabilized soil is very low.

Great care must be observed in removing specimens from the blotter to prevent loss of weight in the specimen due to portions adhering to the blotter. This is particularly true in the untreated specimens which take up water quickly and become soft and noncohesive. Any portion of the specimen remaining on the blotter must be removed and replaced on the bottom of the specimen before each weighing. The necessity for doing this can be avoided by placing a disk of saturated filter paper under the specimen when it is first placed on the blotter and then weighing the specimen with the filter paper each time. The weight of the saturated filter paper is, of course, added to the initial weight of the dry specimen, which weight also includes the cellophane wrapper and the cover.

The stability test apparatus consists of a heavy steel cylinder 2¹⁄₁₆ inches in inside diameter and 4 inches or more high. It is provided with a tool-steel orifice plate at the bottom, the orifice having an area of exactly 1 square inch and beveled downward from the cutting edge. A plunger 2 inches in diameter and a supporting frame complete the apparatus.

To make the test the specimen is placed in the cylinder with the orifice plugged. The equipment is placed in a compression machine and a total load of approximately 3,000 pounds is applied to press the specimen down into contact with the sides of the cylinder and the orifice plate. The plug is then removed, marks are made at ½-inch intervals on the plunger and the load applied at a rate sufficient to move the plunger ½ inch per minute. The load is read as each ½-inch mark is reached, and consequently at the completion of the extrusion of each ½-inch stratum.

The maximum load applied in making the stability test should usually be about 25,000 pounds (approximately 8,000 pounds per square inch), but if desired this may be increased to about 40,000 as the apparatus is designed to withstand this total load. The test is intended to measure resistance to displacement by plastic flow, and a specimen which resists displacement under a load of 8,000 pounds per square inch is virtually a solid and may be so regarded. Test values beyond this limit are not usually significant as an index of plasticity.

Soil-tar mixtures.—Tests for determining the amounts of tar to be used for stabilizing the various types of soil are as yet in a state of development, the most important source of information of this kind being now furnished by studies of experimental road sections. The density tests are largely used for determining the amount of water required in such mixtures.

Road oil and cutbacks.—The amounts of asphaltic oils and cutbacks required to stabilize soils for use in base courses have been determined chiefly from considerable mileages of experimental roads constructed mainly in Missouri.

HIGHWAY EMBANKMENTS

Specifications proposed to the American Association of State Highway Officials for embankment soils contain essential requirements as follows:

Condition 1.—Fills not exceeding 10 feet in height which are not to be placed on steep sloping foundations, and which are not subject to long periods of inundation.

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[5] Soil Stabilization with Emulsified Asphalt. C. L. McKesson. Proceedings Fifteenth Annual Meeting of the Highway Research Board, December 1935.

Liquid limit, not greater than 65.

For soil with liquid limits less than 65, the plasticity index should be not less than that determined by the formula 0.6×liquid limit−9.0.

The soils are rated as to maximum dry weights in table 2.

TABLE 2.—*Soils rated as to maximum dry weights for condition 1*

Maximum dry weight (lb/cubic foot)	Approximate Bureau of Public Roads classification	Rating	Minimum field compaction requirement in percentage of maximum dry weight
			Percent
89.9 and below	A-5, A-8	Unsatisfactory	----
90.0–99.9	A-5, A-8	Very poor	95
100.0–109.9	A-6, A-7	Poor	95
110.0–119.9	A-4	Fair	90
120.0–129.9	A-3, A-2	Good	90
130.0 and above	A-1	Excellent	90

Condition 2.—Fills exceeding 10 feet in height, or which may be placed on steep sloping foundations, or which are subject to periods of inundation.

Liquid limit, not greater than 50.

The soils are rated as to dry weights in table 3.

TABLE 3.—*Soils rated as to maximum dry weights for condition 2*

Maximum dry weight (lb/cubic foot)	Approximate Bureau of Public Roads classification	Rating	Minimum field compaction requirement in percentage of maximum dry weight
			Percent
99.9 and below	A-5, A-8	Unsatisfactory	
100.0–109.9	A-6, A-7	Very poor	100
110.0–119.9	A-4	Poor	95
120.0–129.9	A-3, A-2	Fair	90
130.0 and above	A-1	Good	90

ESSENTIAL FEATURES OF CONSTRUCTION

Subgrades. It has been found economical to separate different classes of materials excavated in grading operations and replace them in the constructed fills in such way that the better materials will be placed next to the pavement. This has been found beneficial for reducing the destructive effects of frost heave in silty soils and the volume change of clay soils.

One of the most successful methods used for overcoming the detrimental heaving due to frost prevalent in the northern part of the United States consists of replacing the material which heaves with selected granular materials. Very fine sands and silts characterized by a lack of cohesion and high water-holding capacity should be replaced generally with the better soils. Such soils are easily identified by visual inspection during the subgrade survey. During the grading operations, the silty soils have been excavated for a depth of as much as 3 feet and the selected material, usually locally available, has been compacted in their place. It has been found necessary to provide proper drainage so that the replaced material does not become saturated.

Several of the States require soils of the A-5, A-6, A-7 and A-8 subgrade groups, which have been found to be inferior materials for the construction of fills and subgrades, to be covered when used as subgrades with better soils. Usually a minimum depth of 1 foot has been required and in many cases, 18 inches to 2 feet.

In some cases plastic soils used for subgrades have been compacted at a controlled moisture content prior to the construction of the surface.

CONSTRUCTION OF STABILIZED ROADS AND BASES WITH GRADED MIXTURES

The essential construction features have to do with the proper mixing of materials, compaction and, in the case of bituminous admixtures, control of moisture content. By road mixing methods, it has been found possible to furnish graded mixtures with a uniformity such that the variation in the plasticity index of the finished road does not exceed 3. It is possible that the same degree of uniformity can be obtained, especially in the more plastic soils, by plant-mixing methods at little or no extra cost. Mixing on the road is facilitated by having the materials dry. It seems that adequate compaction can be attained in chemically treated mixtures with 8 to 12 percent moisture content.

The density of road surfaces can be expected to increase under traffic after construction. In bases an effort should be made to obtain a high degree of compaction during construction if the tops are to be applied soon afterward. It seems that the dry weight per cubic foot of the compacted materials should be at least 130 pounds before the application of the bituminous surface treatment.

When a highway is built on a new location, a sufficiently wide roadbed or subgrade is first constructed, using available materials. Loose, sandy subgrades should be improved by the addition of soil binder; soft, unstable subgrades should be improved by the addition of granular material. The added material should be thoroughly mixed into the subgrade by harrowing or blading, after which the subgrade is brought to true alinement and grade by blading and rolling. Excess subgrade material that is suitable for use as binder for the wearing course can be bladed to the shoulders of the road and left in windrows during the preparation of the subgrade.

When an existing highway is improved, material is added or taken away to obtain the desired width, grade, and alinement. Soil binder or granular material is added in sufficient amount to obtain a stable mixture, and is thoroughly mixed into the subgrade by harrowing or blading.

There are two methods for obtaining the desired mixture—plant mixing and road mixing.

Plant mixing.—The essential requirements for plant mixing are a source of satisfactory binder in close proximity to the supply of coarse aggregate to be used; means for drying and pulverizing the binder; apparatus for measuring the quantities of binder and aggregate to obtain the proper proportions; and equipment for thorough mixing of the combined materials.

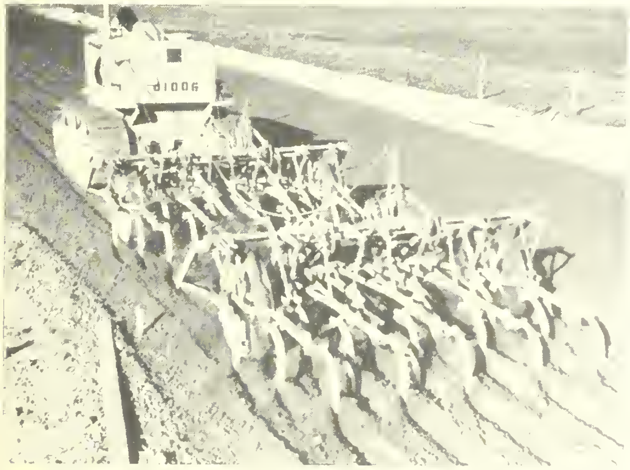
An ideal location for a mixing plant is one near a place where the overburden consists of a clay soil having satisfactory binder properties. This overburden may be stripped and stockpiled in windrows so that it will dry sufficiently to be pulverized under a roller. The clay should then be combined with an aggregate of proper grading in such proportions that the resulting mixture will conform to the specifications. Thorough mixing should be provided by means of a pugmill or rotary-type mixer.

In some plant mixes, bank-run gravel and the existing overburden have been used and the drying and pulverizing steps have been eliminated by passing the combined materials through a rotary screen or over a grill to remove oversize material.

The mixture is hauled to the road, spread, sprinkled, shaped, and rolled. The principal advantage of the plant-mix method is that it effects a thorough mixing of the binder and aggregate.

Road mixing.—The necessary equipment for mixing materials on the road includes tractor-drawn or self-propelled blade graders, scarifiers, harrows, and other apparatus for mixing, moistening, spreading, and compacting the base- and wearing-course materials.

Scarifiers may either be mounted on special four-wheel vehicles or may be attached to the roller. Disk harrows should be so designed that unintended cutting into the subgrade can be avoided. Either the spring- or spike-tooth harrows, that assure positive control of the depth to which the teeth will penetrate, are satisfactory. For mixing and shaping, power-drawn blade graders with a wheelbase of not less than 16 feet are satisfactory.


A Spring-Tooth Harrow Which Permits Positive Control of the Depth of Penetration of the Teeth.

The surfaces of existing roads should be scarified deep enough to eliminate all irregularities of the surface and to permit reshaping to grade. After the loosened material is shaped to grade, the surface should be scarified to a uniform depth. New material, if required, should be spread on the loosened surface in amount sufficient to furnish a compacted layer of approximately 3 inches. If greater surface thickness is desired, additional layers should be constructed.

The material should be mixed, until uniform throughout, by means of harrowing and turning with blade graders. The water required to obtain the proper moisture content in the wearing course should then be supplied in the following manner: The dry mixture of binder and gravel should first be bladed onto the shoulders of the road. The exposed base should then be moistened uniformly using a suitable sprinkler, and approximately one-fourth of the material from the shoulder windrows should then be bladed immediately onto the moistened base. The newly distributed road material should then be similarly moistened and covered as before with one-fourth of the original windrowed mixture, and similar operations repeated until the last fourth of the dry mixture has been placed.

A Pneumatic-Tired Roller Especially Devised to Consolidate Stabilized Soil Surfaces.

Any means of applying adequate load, whether it be a flat-wheeled roller, a crawler-type tractor, truck graders, other trucks, or any vehicular traffic, will assist in the compacting of graded mixtures. After shaping and initial compacting are completed, the road should finally be thoroughly dampened and again compacted.

During the period of final compaction, loose material should be kept distributed over the road surface by means of a road drag or blade grader. The amount of crown maintained by blading operations should be sufficient to provide for the rapid run-off of rain and prevent the formation of pools of water on the road surface. Pitting of the surface is an indication that the crown is too slight.

Chemical admixtures.—In plant mixing, calcium chloride is generally added to the mixture at the plant. In road mixing, calcium chloride is applied in the following manner:

After the new and old soil materials have been spread over the prepared base in sufficient thickness, they should be covered uniformly with flake calcium chloride in the amount of ½ pound per square yard per inch of thickness, with a maximum of 2 pounds per square yard. The spreaders should be capable of spreading the chemical uniformly and in the desired quantities. The calcium chloride is then thoroughly worked into the soil by harrowing and blading until the surfacing material is uniform throughout. Sprinkling and shaping into a finished surface is then accomplished in the same manner as in the case of the untreated roads already described.

Calcium chloride should not be applied during periods of cold, wet weather. The most favorable time for a surface application of calcium chloride is during the drying period just following a rain, after the surface has been bladed to proper smoothness. The moisture in the road at that time hastens solution and penetration. In the absence of rain, application in the early hours of the day, when the air has high relative humidity, is very desirable. Calcium chloride applied just prior to a rain will probably be washed off the road and consequently afford little or no benefit. When blading of the calcium-chloride treated road is required, it should be done after rains or commenced near the end of a rain. At other times, the surfaces may be too hard. For patching holes an admixture of 100 to 150 pounds of calcium chloride per cubic yard of graded soil is recommended.

Need for extensive patching of stabilized soil surfaces during dry weather can be minimized or eliminated by periodic maintenance with light applications of calcium chloride during the dry season.

In the use of calcium chloride as a dust palliative, it has long been known that this material remains effective for much longer periods in gravel having a clay binder than in gravel having a sandy filler. It has also been apparent that the presence of loose gravel tends to shorten the useful life of a treatment. The intermittent movement of such loose gravel by traffic and maintenance operations exposes new surfaces to the air and promotes evaporation.

Sodium chloride may be added at the plant for a plant-mix or to the mixture on the road for a road-mix. Rock salt has been used most generally, but any commercial type of pure salt or salt brine is satisfactory. The coarser grades of rock salt are well adapted for stabilization, as they remain free-flowing and can be more uniformly spread than the finer gradations, which will absorb mositure and cake when stockpiled along the road. The salt should contain not less than 98 percent of pure sodium chloride.

When sodium chloride is applied in the form of brine, any gradations of the salt may be used, since the condition of the salt before it is dissolved is unimportant. The stabilized mixture will remain plastic longer if all of the salt has been dissolved, thus permitting rolling and compacting to be continued over a long period of time with the possibility of attaining greater ultimate compaction. There should be enough water to moisten the soil somewhat above the plastic limit.

Water is usually added by means of pressure distributors, sprinkler wagons, or gravity tank trucks. It is preferable to make several trips over the section under construction, sprinkling lightly on each trip. This gives the moisture a chance to penetrate sufficiently and does not create a sloppy condition on the surface. The amount of salt used varies somewhat in the different States. The usual amount is about 2 pounds per square yard per 3-inch thickness of road constructed.

During prolonged dry periods, the fine aggregate may wear off of a sodium chloride treated surface, leaving the larger aggregate particles protruding. This condition can be corrected by covering the surface to a depth of about ½-inch with properly proportioned soil mixtures containing about 100 pounds of sodium chloride per cubic yard. When damp the mixture makes a satisfactory bond with the worn surface without scarification.

If, during periods of rainfall, the top eighth or quarter of an inch of stabilized roads becomes plastic, crushed aggregate should be added to the surface in amounts of about 50 to 70 tons per mile. This aggregate should have a maximum size of about ½-inch and should be free from dust and sand. The material should be spread over the surface and kept uniformly distributed as long as the surface is wet. The thin surface layer of fine material acts as a cementing agent for the coarse aggregate that has been added. This results in a satisfactory driving surface, adds some additional thickness to the road, fills up any depressions that may have developed, and reduces the tendency for pot holes to form.

CONSTRUCTION OF CEMENT STABILIZED BASES

The construction of a cement stabilized base, recently developed experimentally in South Carolina, is summarized as follows:

1. *Pulverizing the soil.*—The soil on the road was effectively scarified to the desired depth with a tooth-scarifier and then pulverized with a disk harrow assisted by a sheepsfoot roller where dry, hard lumps of soil were encountered.

A Tractor-Drawn Disk Harrow.

2. *Spreading cement.*—The cement was spread by means of a sand spreader; which was attached to and pulled by loaded dump trucks containing a known quantity of loose cement. Considerable time was lost in spreading as each time a truck was emptied, it was necessary to unfasten the spreader and attach it to the next loaded truck. The moisture content of the soil should be below the optimum required for compacting, as high moisture contents cause the cement to form into balls.

3. *Mixing cement with dry soil.*—A disk harrow was first run slowly over the freshly spread cement in order to mix it partially with the loose soil. The cement flowed into the furrows cut by the disks, and if the disks were allowed to cut below the depth to be treated, the cement would flow into the bottom of the furrows and be lost, as the other mixing equipment would not cut into the hard subgrade. Mixing was completed by using a road machine and multiple-blade drag simultaneously. The drag followed the road machine and mixed the roll of material turned up by the road machine blade. Mixing in this way was first performed on the edges and continued through the middle. Approximately 12 trips of each machine were required for the dry mixing.

4. *Application of water.*—Water was applied with an asphalt distributor of 1,000 gallons capacity. This quantity of water was spread uniformly over the surface in one trip. The distributor was filled at each end of the section by feeder trucks.

5. *Mixing.*—After each application of water the materials were partially mixed with a 20-inch disk harrow pulled as rapidly as possible by a 75-horsepower crawler-tread tractor. For most efficient mixing, it was found that the disk harrow should be operated at a high speed so that the disks would throw the material instead of merely cutting into it. On sections 6 inches thick, it was necessary to store approximately ⅓ of

the dry mixture on the shoulders and apply the mixing water to that remaining in the bottom layer, then bring in the stored material from the shoulders and mix water with it. In order to wet the mixture efficiently at the edges of the road, approximately 3 feet of the mixture near the edges was pulled toward the center of the road when the stored material was brought in from the shoulders. After the water had been mixed with this material, the mixture was spread back to the edges before compaction was started. The entire section was then loosened to the bottom with a disk harrow so that the sheepsfoot roller could begin compaction at the bottom.

6. *Compaction.*—The sheepsfoot roller used for this operation was pulled by a tractor which was operated slowly for the first few trips so that the feet of the roller could penetrate the mix easily and compact the material from the bottom upwards. The roller was then operated at a higher speed in order to obtain the benefit of dynamic forces in the compaction. It usually required about 3½ hours for one roller to work itself to the surface of an 800-foot section of road. It was found that the moisture content of the soil below the treatment and of that on the shoulders must be at the optimum or below to obtain satisfactory compaction. The operation of the sheepsfoot roller on soft subgrades produces numerous fine cracks throughout the compacted surface. Usually about 1 inch of loose material remained on the surface when the sheepsfoot roller had compacted the mix as much as possible, and this loose material was shaped with a road machine to the correct cross section and rolled with loaded trucks.

7. *Finishing.*—The finishing operation consisted of blading the surface to the correct grade and cross section with a hand-operated road machine. This operation determined the riding quality of the section and was done by very skillful operators. This work was partly done during compaction and completed before the trucks had finished rolling the surface. It was found that the road machine blade would not cut the surface of the hardened base the day after it was constructed, and, therefore, it was necessary that the surface be finally shaped as soon as possible after compaction.

CONSTRUCTION OF TAR STABILIZED BASE

The construction of an experimental road in South Carolina in which coal tar was used as a stabilizing agent,[6] has been described as follows:

The soil in this project, practically all of which passed the No. 10 sieve, contained about 16 percent silt and 28 percent clay (by elutriation). Other characteristics were approximately as follows: Liquid limit 28, plasticity index 13, field moisture equivalent 19, shrinkage limit 15, specific gravity 2.57, and volume change 23.

The total amount of tar used was 3.73 gallons per square yard of base 6 inches in compacted thickness. Of this quantity of tar 1.48 gallons was 4 to 8 viscosity (Engler) and the remainder 26 to 36 viscosity (Engler). The lighter tar was applied and thoroughly mixed with the soil before the heavier tar was used.

It was found that small lumps of clay soil did not cause trouble if the lumps were surrounded by material impervious to water and protected from traffic.

It was noticed that soil mixed with low-viscosity tar alone repelled water so completely that rain for from 2 to 9 days did not penetrate windrows of the mixture more than ¼ inch.

Three different sections contained, respectively, 2.9 percent, 7.5 percent, and 5.8 percent moisture when mixed with the low-viscosity tar. It was noticed that the section with 7.5 percent moisture had a "slightly grayish tinge" during mixing rather than the "deep black cast" that had been observed on the section containing 2.9 percent moisture. Furthermore, it appeared to be lean and dusted under traffic. It was thought that the differences in appearance and behavior of these two sections undoubtedly were due to the fact that the greater moisture content of the second section caused the tar to penetrate the soil, and particularly lumps of soil, more completely in that section.

The equipment used on this project consisted of a disk harrow, tractors and blade graders, retread machine, pressure distributor, and trucks.

Soon after this project was completed the heavier tar was applied to the surface at the rate of 0.26 gallon per square yard and covered with sand. No other cover has been applied since completion 2 years ago.

CONSTRUCTION OF EMULSION STABILIZED BASE

The construction of a soil surface with admixture of bituminous emulsion [7] has been described as follows:

Laboratory studies indicated that the natural adobe clay soil could be stabilized to a depth of 3 inches with about 3 gallons of 55 percent emulsified asphalt. By adding 2 parts of quarry waste to 1 part of the clay, stabilization was accomplished with 1 gallon of emulsion per square yard, a saving after paying for the quarry waste of about 5 cents per square yard. The construction methods were as follows:

The quarry waste was uniformly spread on the clay subgrade to a loose depth of 1¼ inches and scarified into the natural clay. The loose mixture was 4½ inches thick. The clay and quarry waste were well mixed by harrowing.

The soil mixture was first dampened with about 2 gallons per square yard of water and the water was mixed in by harrowing. The emulsified asphalt was then applied, after dilution with 4 parts of water. The diluted emulsified asphalt was applied in 6 applications. After the first application, the mixture was harrowed and the top inch and a half of the mixture was bladed to the sides. These operations were repeated after the next 2 applications. The soil mixture was then in 2 windrows on the sides and was distributed back over the road in 3 spreads with application of dilute emulsion to each.

To obtain thorough mixing, the soil-emulsion mixture was again bladed into windrows, then brought back in thin layers, and finally shaped to the required cross section. The tractor used to pull the mixing equipment had by this time fairly well compacted the road surface.

One gallon of emulsion and 6.2 gallons of water were used per square yard, including the water used in the preliminary dampening operation. This completed the soil stabilization and there remained only the surface treating.

The surface was then given an application of approximately ¼ gallon per square yard of the emulsion, diluted with an equal part of water and immediately covered with 40 pounds per square yard of ½ to ¼-inch crushed stone. This was followed by rolling of the surface which embedded the crushed stone in the stabilized soil mixture. A final application of 0.64 gallon per square yard of undiluted emulsified asphalt was then spread and immediately followed with 15 pounds per square yard of ¼ inch to No. 10 sieve crushed stone, followed by brooming and rolling.

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[6] Soil Stabilization with Bituminous Materials in South Carolina, J. S. Williamson, Convention Proceeding 1935–36 of the American Road Builders' Association.

[7] New Developments in Asphalt Construction, C. L. McKesson, Association of Highway Officials of the North Atlantic States, New York City, February 26, 1937

STABILIZATION WITH CUTBACKS AND ASPHALTIC OILS

An experimental stabilized road in Jackson County, Missouri, was compacted by traffic. Asphaltic materials used as stabilizers were introduced at desired depths below the surface of the loose road mix by means of a specially constructed "sub-oiler."[8] This device is essentially a tooth scarifier having an oil line attached to the back of each tooth and running nearly to its point.

The theory underlying this procedure has been that as moisture evaporates from the overlying soil, the moisture films will be replaced by liquid asphalt. Roads consisting of heavy, sticky clay or gumbo types of soil became water resistant after treatment in this way and were unaffected by rains.

Another method for incorporating the asphalt into the soil was tried in Missouri during 1935.[9] The clay surface was scarified and the loosened material windrowed and mixed with the desired percentage of bituminous material by means of a traveling mixing plant.

Varying quantities of coarse aggregates were added to the soil to obtain information as to the physical and economical effects of the use of such materials. The conclusion reached was that percentages of aggregate may be varied to obtain economy dependent upon the relative costs of aggregate and bituminous materials, since the addition of material larger than the No. 40 sieve permits a proportional decrease in oil.

About 35 gallons of water per cubic yard of mixed material were added to facilitate mixing in the pug mixer. The sections that had time to dry prior to freezing carried traffic with little or no sign of distress, although there was some evidence of a sponginess or rubbery action indicating that future additional consolidation would and should take place.

SUMMARY

The study of road subsoils has been utilized in the following ways in the United States:

1. The subgrade survey and subgrade classification has been used in the location and design of highways and particularly in the prevention of (*a*) frost heaving and (*b*) excessive pavement breakage and faulting.
2. The testing methods described have been employed in the design and construction of stabilized road surfaces and base courses of (*a*) the graded type and (*b*) the fine-grained type in which an adhesive binder such as portland cement or bitumen has been used.
3. The density tests as described have been used for the selection and control of construction of fills.
4. Special investigations have been made for the solution of foundation problems.
5. The problem of constructing stable bases for light bituminous surface treatment has been extensively investigated by the State Highway Department of the State of Missouri. This work has included mechanical stabilization (clay-aggregate stabilization), portland cement, bituminous, and mixed bituminous and mechanical stabilization.

(*a*) It is indicated that all the methods employed may be successful but the relative economy of various methods depends on local conditions associated with the project. Mechanical stabilization assumes the availability of an ample aggregate supply. Economy limits cement stabilization to soil types requiring a relatively small amount of cement. In other cases, bituminous treatments will generally prove more economical and adaptable.

(*b*) In bituminous stabilization three methods of treatment—machine mixing, sub-oiling, and road-mixing—have all been successfully employed. The sections of road-mixed material are of too recent date to be fairly compared with the other sections, but the results obtained so far give no indications of inferiority.

(*c*) Machine mixing as carried on in Missouri produced complete diffusion of the treatment. The presence of fine material in the mix promoted the formation of a clay emulsion as indicated by the appearance and action of the material discharged by the pug mixer. Tar-treated material in particular instances has not shown any such action but diffusion and coating have appeared complete.

(*d*) Asphalt emulsions because of their ability to diffuse through the roadbed materials worked to best advantage in conjunction with the sub-oiling operation. However, cheap road oils of the slow curing grades were successfully used when dispersed by supplementary blade mixing after the application of oil by means of the sub-oiler.

(*e*) The necessary quantity of portland cement for use is apparently easily determined by use of the Proctor density tests in combination with durability tests consisting of freezing and thawing and alternate wetting and drying tests.

(*f*) In the case of bituminous treatments there is apparently a rather flexible range of quantity of treating agents which may be successfully used. This quantity ranges from approximately 6 to 10 percent by weight of the soil fraction. As judged by the behavior, mixtures near the upper limits of the range mentioned appear to be more critical than those near the lower limits, no radical failures being noted in sections approaching excessive leanness whereas an excess of bitumen has immediately produced plasticity and loss of stability.

(*g*) Under the conditions used, and based on the length of service thus far observed, no appreciable difference has been noted in the stability produced, or the durability obtained, with the various bituminous agents used. Unless additional length of exposure develops differences in durability or behavior, economy in cost of treating agents will probably be the governing factor in selection.

(*h*) One of the results of planning and carrying out the research projects is the conclusion which immediately becomes apparent, that there is little or no fundamental information available on which to base thickness design.

ACKNOWLEDGMENTS

The Principal Reporter acknowledges the very valuable data and papers furnished him for the preparation of this report. The contributors were Miles D. Catton, Development Department, Portland Cement Association; W. W. Johnson, State Highway Commission of Kansas, and F. V. Reagel, Engineer of Materials, Missouri State Highway Department.

The Principal Reporter quoted freely from all the information furnished.

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[8] Soil Stabilization Work in Jackson County, Missouri: Frank V. Gilmore: Roads and Streets, 68. 313–316, 1935.

[9] Use of Cutbacks and Road Oils in Soil Stabilization, F. V. Reagel and R. C. Schappler, Proceedings of 16th Annual Meeting Highway Research Board, vol. 16, p. 359, 1936.

(Continued from page 111)

light from each type of pavement tested both in a wet and a dry condition.

Attention is called to the fact that the wetting of the surface of the bituminous macadam and the open-type asphaltic concrete has little effect upon glare. This is borne out by driving experience over these types of surfacings. On the contrary, smooth and fine sandpaper-textured surfaces show a greatly increased glare when wet. Such surfaces, when wet, are covered with a film of water which becomes the reflecting surface, while the irregular surfaces, having pits or indentations ¼ inch or more in depth, break up the water film. It is thus apparent that the relative glare from wet surfaces is in the main a function of the texture of the surface, and the least glare is given by those surfaces which are so deeply indented that the water will drain off with sufficient rapidity to prevent the flooding of the entire surface and the formation of a water film which acts as a mirror in reflecting the lights of oncoming vehicles.

ACKNOWLEDGMENTS

The Principal Reporter acknowledges the very valuable data furnished him for the preparation of this report. The contributors included R. A. Moyer, Associate Professor of Highway Engineering, Iowa State College; H. E. Phelps, Professor of Highway Engineering, State College of Washington; G. F. Schlesinger, Engineer-Director, National Paving Brick Association; Bernard E. Gray, Chief Highway Engineer, The Asphalt Institute; A. A. Anderson, Manager Highways and Municipal Bureau, Portland Cement Association; T. S. O'Connell, State Highway Engineer, Arizona State Highway Department; W. C. Ross, Assistant Construction Engineer, Arkansas State Highway Commission; J. E. Maloney, Assistant Engineer, Colorado State Highway Department; A. B. Nuss, State Highway Engineer, Kansas State Highway Commission; G. H. Delano, Chief Engineer, Massachusetts Department of Public Works; R. A. Harris, Chief Engineer, Mississippi State Highway Department; C. S. Mullen, Chief Engineer, Virginia Department of Highways; E. L. Roettiger, State Highway Engineer, State Highway Commission of Wisconsin; J. G. Smith, District Engineer, Wyoming State Highway Department; and G. S. Paxson and J. D. Everson of the Oregon State Highway Department.

The outstanding contribution of Professor R. A. Moyer of the Iowa State College forms the basis of the first section of this report. The Principal Reporter quoted freely from all the information furnished.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JULY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 18,600	$ 9,300		$ 7,180,388	$ 3,583,440	290.6	$ 2,755,844	$ 1,374,955	121.8	$ 3,782,495
Arizona	1,038	747		1,710,711	1,294,548	91.9	227,955	146,378	14.9	1,734,296
Arkansas				914,892	904,226	74.5	2,046,596	2,043,355	125.9	2,224,424
California	638,168	336,189	17.1	10,947,077	5,743,959	200.5	3,742,136	1,999,794	91.8	1,912,982
Colorado	268,441	142,332	13.1	2,413,481	1,326,808	75.4	708,171	393,822	16.6	2,853,299
Connecticut				1,189,000	579,991	13.1	405,970	201,510	3.6	1,531,972
Delaware	54,130	27,065		782,314	385,679	22.3	480,998	241,791	1.4	1,184,277
Florida				2,942,251	1,471,126	61.2	2,666,856	1,058,828	33.9	2,815,962
Georgia	886,329	443,164	50.9	6,592,734	3,296,367	307.1	2,078,040	1,039,020	109.0	5,518,798
Idaho	267,078	158,698	16.5	1,973,726	1,176,262	184.0	581,165	345,748	7.5	1,227,141
Illinois	317,000	179,284	10.3	10,231,338	5,110,255	217.4	4,025,790	2,012,895	124.9	3,201,711
Indiana	1,000,000	500,000	19.2	5,032,417	2,510,140	131.5	2,120,340	1,059,950	39.3	2,511,545
Iowa	701,727	319,146	17.1	6,993,809	3,137,683	225.3	2,158,879	1,014,673	76.7	1,162,647
Kansas	13,330	6,665	.5	4,871,879	2,392,192	320.1	5,555,486	2,777,727	631.5	3,120,023
Kentucky	436,832	218,416	32.3	4,908,412	2,454,206	157.4	3,301,496	1,638,006	76.5	2,114,640
Louisiana	256,047	128,000	9.3	11,647,244	2,221,315	46.2	2,155,846	971,631	41.5	2,399,773
Maine	452,048	226,529	10.3	2,152,782	1,069,101	48.5	1,077,544	538,771	18.3	415,972
Maryland	54,000	27,000	1.2	1,937,567	967,021	29.5	1,230,411	602,830	18.1	1,992,916
Massachusetts	122,568	61,284	.8	2,675,881	1,337,937	9.9	580,222	290,109	8.7	2,659,435
Michigan	687,200	343,600	25.4	6,335,138	3,097,169	128.2	2,495,890	1,244,000	53.3	2,455,712
Minnesota	236,204	106,916	5.9	5,144,493	2,546,079	250.4	2,534,846	1,262,167	130.0	2,935,473
Mississippi	121,400	60,700	6.2	7,255,460	3,091,525	325.5	2,946,200	777,840	130.1	3,223,880
Missouri	275,580	137,790	13.2	4,850,897	2,395,786	125.2	4,535,941	1,929,031	163.0	3,635,997
Montana	351,319	197,613	18.3	1,396,200	784,993	52.5	252,912	142,262	8.4	4,481,815
Nebraska	419,442	209,721	63.7	5,865,800	2,924,429	491.3	3,874,158	1,302,084	166.3	2,656,210
Nevada	647,806	561,764	65.8	830,919	719,392	60.5	515,844	442,570	59.5	1,366,464
New Hampshire	175,316	86,246	5.8	983,694	490,088	15.1	171,212	85,534	5.1	1,162,941
New Jersey				2,852,361	1,424,918	21.2	211,060	104,010	.5	2,672,543
New Mexico	381,352	232,583	46.3	1,955,111	1,292,584	164.7	820,802	500,603	53.9	852,860
New York	697,231	344,860	9.8	16,061,399	7,860,712	283.8	3,827,230	1,455,115	54.8	3,570,039
North Carolina	765,500	382,750	59.4	7,150,106	3,447,615	317.9	1,196,893	577,116	70.3	2,899,077
North Dakota				2,565,531	2,487,301	160.5	478,270	420,003	59.4	3,825,290
Ohio	1,460,130	728,110	19.2	8,157,002	4,050,010	82.7	3,748,010	1,863,673	25.4	7,206,088
Oklahoma	948,402	499,098	39.4	4,200,204	2,176,400	152.3	2,847,939	1,495,258	93.9	3,214,684
Oregon	250,986	109,495	1.1	2,245,956	1,331,925	85.0	703,886	429,550	15.7	2,129,944
Pennsylvania	1,894,299	946,100	31.7	5,418,749	2,687,106	82.0	5,579,134	2,788,573	57.8	4,410,663
Rhode Island				1,085,102	542,551	14.8	360,400	180,200	3.2	1,021,049
South Carolina	461,039	210,900	36.2	5,105,356	2,259,535	206.0	951,695	429,290	52.9	1,878,384
South Dakota	128,610	71,130	6.0	3,731,162	2,066,250	370.8	1,214,550	679,480	92.3	3,340,076
Tennessee	385,560	192,780	13.0	5,184,534	2,592,267	153.2	913,400	456,700	19.7	4,747,348
Texas	2,617,918	1,299,835	166.3	10,306,467	5,105,482	589.3	4,862,157	2,377,080	225.1	7,575,569
Utah	3,300	2,360		1,294,470	927,400	108.4	581,880	415,673	36.1	1,459,499
Vermont	147,643	67,887	3.1	1,447,682	659,520	39.4	241,698	105,199	6.9	292,304
Virginia	1,091,814	545,907	41.0	5,008,991	2,502,515	167.7	1,916,636	956,033	47.8	1,033,162
Washington	7,917	3,600		4,573,308	2,386,214	75.3	615,217	322,300	13.9	1,296,312
West Virginia	115,250	92,300	9.2	1,705,746	1,131,733	45.8	1,362,406	771,578	34.5	2,220,246
Wisconsin	669,468	321,477	17.0	6,917,676	3,208,835	193.1	2,011,961	878,880	62.7	1,879,276
Wyoming	210,330	127,920	27.4	1,884,027	1,157,562	202.3	336,631	207,960	66.2	798,986
District of Columbia										
Hawaii	103,405	51,703	1.9	795,285	394,357	16.5	776,440	380,075	8.8	1,198,792
Puerto Rico				1,128,739	562,390	20.5				671,985
TOTALS	20,741,757	10,718,964	930.9	220,535,468	109,266,899	7,508.3	90,785,043	44,731,630	3,379.4	126,476,976

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JULY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 459,700	$ 229,750	30.2	$ 192,005	$ 94,750	13.7	$ 728,472
Arizona				196,056	129,008	11.4	92,929	63,940	7.2	463,534
Arkansas				19,426	12,863					851,245
California	$ 146,008	$ 84,753	16.7	827,590	454,517	41.0	738,150	395,006	28.6	910,656
Colorado				688,486	381,243	33.5	141,044	76,430	14.8	463,895
Connecticut							76,090	38,025	1.3	280,253
Delaware							21,700	10,850	4.0	236,025
Florida				20,122	10,061					664,791
Georgia	71,680	35,840	8.4	277,418	138,709	36.4	487,417	243,708	62.9	840,308
Idaho	105,529	52,041	8.0	306,022	133,494	33.6	12,569	5,080	3.5	280,884
Illinois	95,000	47,500	10.3	1,269,832	580,916	88.8	1,279,800	639,900	94.9	758,599
Indiana				772,200	305,700	77.9	746,153	369,800	71.7	566,938
Iowa										1,298,449
Kansas	43,670	21,835	.5	63,694	31,847	.7	163,886	81,942	40.1	1,183,445
Kentucky	109,248	33,608	16.4	662,226	193,254	53.4	1,445,259	407,476	127.5	180,088
Louisiana				21,670	10,835		498,299	191,750	39.0	521,551
Maine	122,600	61,300	8.4	198,936	99,468	11.2	201,000	100,500	11.5	67,305
Maryland				6,264	3,132					409,344
Massachusetts				5,300	2,650					643,750
Michigan				23,362	11,681		764,500	382,250	49.5	1,141,608
Minnesota				381,352	171,027	37.1	27,946	11,610	8.7	1,196,943
Mississippi										888,927
Missouri	97,410	47,800	17.4	250,250	124,735	19.5	526,360	183,745	61.9	708,808
Montana				13,983	7,865					1,027,170
Nebraska	13,984	6,992	2.2	435,186	217,593	60.3	250,776	116,812	45.7	654,655
Nevada	57,038	49,463	6.3	199,291	172,542	27.9	139,080	120,539	19.9	148,900
New Hampshire				208,761	103,578	4.5	39,820	19,880	1.5	123,417
New Jersey							181,060	86,875	2.5	585,898
New Mexico	102,849	62,727	10.7	438,545	267,466	20.0				476,579
New York	416,600	208,300	26.7	1,757,260	878,630	116.9	801,000	341,350	34.2	930,181
North Carolina				866,220	433,110	94.2	73,044	36,500	3.4	611,983
North Dakota				33,860	18,135	5.5	149,990	80,332	29.5	688,785
Ohio				184,400	92,200	3.8	11,100	5,000	.9	1,734,441
Oklahoma				84,588	45,008	4.1	486,490	248,819	45.7	891,088
Oregon	74,302	44,310	11.5	256,003	155,342	24.9	179,494	100,610	24.3	452,860
Pennsylvania	215,152	107,576	15.5	1,303,619	632,461	79.6	792,965	385,994	51.1	989,857
Rhode Island	21,730	10,865	.6	112,106	56,053	5.5				120,973
South Carolina				514,080	219,862	57.8	469,018	197,754	53.8	261,064
South Dakota				11,300	6,250					816,436
Tennessee				243,946	121,973	12.6	442,580	154,230	19.9	783,287
Texas	131,830	65,390	27.2	1,801,796	833,699	221.2	1,194,610	485,190	151.5	1,740,246
Utah	50,570	25,000	3.8	202,455	115,790	18.8	250,575	120,530	27.5	289,050
Vermont	140,880	60,675	8.3	59,176	27,178	3.3	55,100	15,247	2.2	106,440
Virginia	66,100	33,050	6.9	535,227	265,398	44.8	529,390	202,909	33.9	390,642
Washington				423,079	222,378	31.6	474,906	249,800	50.6	258,378
West Virginia				208,000	104,000	16.5	118,200	59,100	9.4	386,574
Wisconsin				641,365	304,965	24.6	310,170	121,580	15.0	785,683
Wyoming	221,060	136,590	22.9	157,360	97,230	20.9	61,500	38,000	5.1	313,373
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico				244,000	121,950	13.7				124,925
TOTALS	2,303,240	1,195,615	228.7	17,441,762	8,573,671	1,390.1	14,425,975	6,483,813	1,268.4	31,197,453

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TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

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STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JULY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama						$ 541,798	$ 541,724	7			$ 595,610	$ 595,110	6		6	$ 848,685
Arizona						4,718	4,718									625,495
Arkansas						279,639	278,482	7			348,844	348,462	6			1,131,825
California						1,206,964	1,206,389	3	4		348,426	338,944	3			2,154,876
Colorado	$ 18,855	$ 18,855	1		1	19,348	19,348				58,956	55,196	1		3	1,194,643
Connecticut																844,490
Delaware											77,270	77,270			23	416,480
Florida						10,616	10,616				178,800	178,800	1			1,216,381
Georgia						18,346	18,346				27,460	27,460	1			2,371,581
Idaho	87,325	87,652	1			12,342	12,342				106,037	99,220	2			623,656
Illinois						814,675	814,675	4			775,080	775,080	7	1	29	3,614,888
Indiana	97,500	97,500		2		687,000	660,100	5	3		720,260	683,600	1	1	86	971,506
Iowa	17,747	16,100			5	985,329	931,300	12	1	4	247,051	230,400	1		12	1,384,766
Kansas	112,408	112,408	3		2	460,089	458,089	8		2	566,699	566,699	8		3	1,435,607
Kentucky						145,000	145,000	1			374,266	374,266	2	1	31	1,294,779
Louisiana						146,486	146,478	2			327,021	293,050	6			1,137,543
Maine						293,297	293,297	3	2		124,010	124,010	2		1	273,896
Maryland						64,586	64,586	1								962,247
Massachusetts						70,420	70,420		1		162,480	162,480	1			1,680,388
Michigan	110,000	110,000	1	1		568,152	568,152	3		45	759,200	759,200	8			1,847,833
Minnesota						720,196	720,196	2	6	1	100,140	100,140	1			1,601,146
Mississippi						356,800	356,800	4								1,227,351
Missouri	19,380	19,330	1			239,130	239,130	3			149,140	149,140	2			2,617,603
Montana						365,654	360,772	4			276,614	276,614	2			687,085
Nebraska						311,783	311,783	11			13,880	13,880	2			1,438,747
Nevada	5,626	5,626			2	91,561	91,561		2		52,574	52,574		1		308,880
New Hampshire						69,138	68,831		1							424,919
New Jersey						210,005	204,779	2	1							1,765,478
New Mexico						122,441	122,441	4			77,677	77,677		1		651,373
New York						1,831,201	1,825,600	8	4	2	440,868	439,518		3		4,504,811
North Carolina						414,060	414,060	3	2		202,950	202,950	1	4		1,827,384
North Dakota						545,688	545,688	1	1		62,630	62,630	1	1		970,903
Ohio						32,120	32,120				267,110	267,110	4			3,929,938
Oklahoma						17,343	17,343									2,264,166
Oregon	95,263	94,614		2		220,687	220,687	1			9,754	9,754				650,276
Pennsylvania						299,116	283,910	2								5,448,625
Rhode Island						227,553	227,553	1			67,326	67,326		2		198,871
South Carolina						73,933	51,147	1	1		397,131	397,131	1	1	34	1,040,824
South Dakota						125,508	125,508	1	1		258,290	256,150	3	1	10	936,215
Tennessee						14,381	14,381				16,450	16,450		1		1,861,202
Texas	38,798	38,100	2			598,642	598,642	6			402,850	402,515	3	2		4,301,811
Utah	1,760	1,760			1	121,173	121,173	3			25,400	25,400	1		3	424,655
Vermont	151,590	151,590	3	1		85,148	75,868	3	1		18,220	18,220			7	233,875
Virginia	18,730	18,730		1		279,985	279,985	12	1		385,931	385,931	4		8	1,111,384
Washington						419,518	417,418	5	2	2	382,348	382,348	3	2	2	584,033
West Virginia						369,189	368,169	4			87,870	87,870			5	868,059
Wisconsin						922,873	911,633	8			207,138	175,000	2			1,386,876
Wyoming						159,644	159,644		2		14,530	14,530			6	502,556
District of Columbia																325,431
Hawaii						33,230	33,230	1			163,920	163,920	2	1		296,600
Puerto Rico						61,900	61,550	2			151,661	150,580	2			518,170
TOTALS	775,482	772,315	12	7	11	15,668,405	15,505,664	148	36	56	10,029,372	9,884,605	90	23	269	70,940,812

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 7 SEPTEMBER 1938

EAST PORTAL OF TOOTH ROCK HIGHWAY TUNNEL, OREGON

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - See page 2 of cover for prices

PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 7 September 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

HIGHWAY TUNNELS IN WESTERN STATES

BY THE REGIONAL OFFICE, UNITED STATES BUREAU OF PUBLIC ROADS[1]

TUNNELING is one of the oldest construction activities of man. The Assyrians constructed a tunnel under the Euphrates River with a cross section 12 feet wide by 15 feet high. The Romans employed tunnels in their highways. Near Naples a 3,000-foot tunnel was excavated and the lighting problem was given skillful attention. The width of the tunnel was about 21 feet throughout, but the height was increased from 25 feet at the center to 75 feet at the portals, thus to a degree aiding its illumination in daylight. In the Middle Ages tunneling continued in fortification works and in aqueducts, of which the Languedoc and St. Quentin canals in France are outstanding examples.

Modern times have witnessed the construction of a great many railroad and water tunnels. The outstanding railroad tunnel is the Simplon connecting Italy and Switzerland through the Alps, and is 12.4 miles long. It was bored under great difficulties, one of the main obstacles being the heat that resulted from the excessive depth of cover. Other important railroad tunnels are the Mount Cenis, France to Italy; St. Gothard, in Switzerland; Hoosac, in Massachusetts; Rogers Pass, in British Columbia; and the Moffat, in Colorado.

The greatest advances in modern tunneling methods were achieved through the introduction of compressed air and the development of high explosives.

Modern traffic demands highways that can be traveled with speed and safety. In the mountainous parts of the west provision is being made for speeds of 45 to 50 miles per hour, and economy in design has resulted in the construction of some 35 tunnels. Table 1 gives dimensions, cost, and design details. Figures 1 and 2 illustrate typical cross sections of these tunnels.

Ventilation problems have not developed in most of these tunnels, as nearly all of them are less than 1,000 feet long. Consideration had to be given to ventilation in the East Rim Road tunnel in Zion National Park, 5,613 feet long; in the Wawona tunnel in Yosemite National Park, 4,233 feet long; and in the Broadway Low Level tunnel, between Alameda and Contra Costa Counties, Calif., 3,203 feet long.

The numerous galleries introduced for ventilation in the East Rim Road tunnel served the further purpose of affording the traveler incomparable vistas. These galleries solved the ventilation problem, but with additional cost for heavier lining. The faces of the cliffs in Zion National Park are vertical, and are of extreme height. They are composed of sandstone in horizontal beds. These faces undergo extreme temperature changes and constantly expand and contract, thus developing temperature relief joints. These joints or cracks are both normal and parallel to the faces of the cliffs and extend back some distance from the face. A tunnel located well back from the face would undoubtedly be in entirely self-supporting rock, requiring less lining but more artificial ventilation.

The Tooth Rock tunnel in Oregon, is a model in landscape finish and lighting. The four-lane tunnel on the north approach to the Golden Gate Bridge, and the double-deck, six-lane Yerba Buena Island tunnel connecting the San Francisco-Oakland Bay Bridges are interesting to American engineers because the German system of headings was used in their excavation. (See fig. 3.)

Operations in the Broadway tunnel were of special interest. During construction, temporary timber sets that gave signs of distress, were reinforced solidly for full depth by cement mortar applied by means of a pneumatic cement gun.[2] Louvers introduced in this project for light diffusion by forming a transition at the portals were also unique.

The Willamette tunnel in Oregon, while of moderate length, is bored upon a curve of 4°36′, the curved inner sidewalk being taken into consideration in providing adequate sight distance. This design differs from the usual practice in the west of locating tunnels on tangents, although in the Broadway tunnel reverse curves are incorporated at the ends in order to spread the spacing of the bores from 15 feet at the portals to 100 feet for the major portion of their lengths. The spacing is essential in twin bore tunnels as a safety provision in blasting during construction. Steel lining for highway tunnels was first used in the west in the Ventura-Maricopa tunnels.

TUNNELS HAVE SEVERAL ADVANTAGES OVER OPEN-CUT CONSTRUCTION

It has been the policy of the Bureau to consult with a geologist regarding the rock formations that can be expected to be encountered in the construction of major tunnels. The geologists' advice should be followed as to the extent and character of preliminary exploration work. On a few tunnels the practice has been to open the faces as a part of the location work, and in one instance short preliminary headings were excavated to determine the uniformity of the material.

Where the geologist is in doubt, a preliminary classification using the seismographic method is warranted. The obtaining of seismographic profiles should reduce the number of exploration borings to a minimum.

The question of whether to construct a tunnel or an open cut at a particular location should be decided only after careful study of the probable costs of constructing and maintaining each. Three principal advantages of tunnels over open cuts are:

1. Tunneling generally enables better location and design by reducing curvature and length and hence contributes to highway safety.
2. If special facilities for ventilation and lighting are not required, tunneling reduces maintenance costs and in any event practically eliminates snow removal costs.
3. Tunneling reduces scar (a landscape item), and eliminates erosion.

The necessity of portal structures is a major factor in determining the minimum length of tunneling where critical depths of cover are encountered. Assuming the average cost of portals to be $4,000 each, with the critical depth of cover between 80 and 100 feet the economical minimum length of tunnel, depending upon average excavation and lining costs, would be in excess of 80 feet.

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[1] This report was prepared by engineers of the regional office of the Bureau in San Francisco, Calif., with special assistance as noted.

[2] "Pneumatically applied mortar" is a mixture of portland cement and sand, mixed dry in proportions of 1 sack of cement to 4 cubic feet of sand. It is forced by air through a flexible tube to a nozzle where the mixture is hydrated and discharged by pneumatic pressure against the surface being treated.

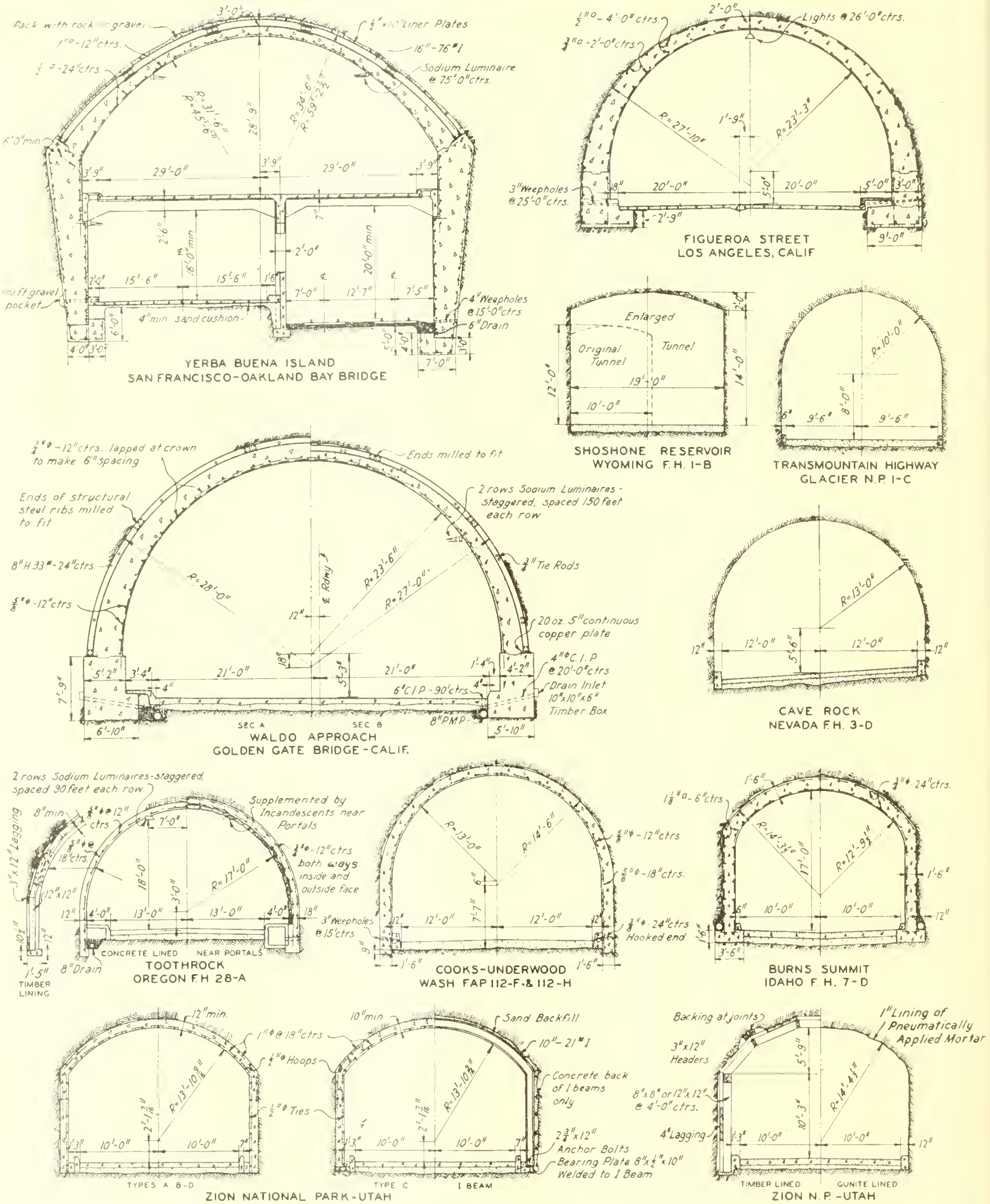

FIGURE 1.—CROSS SECTIONS OF SEVERAL HIGHWAY TUNNELS CONSTRUCTED IN THE WESTERN STATES.

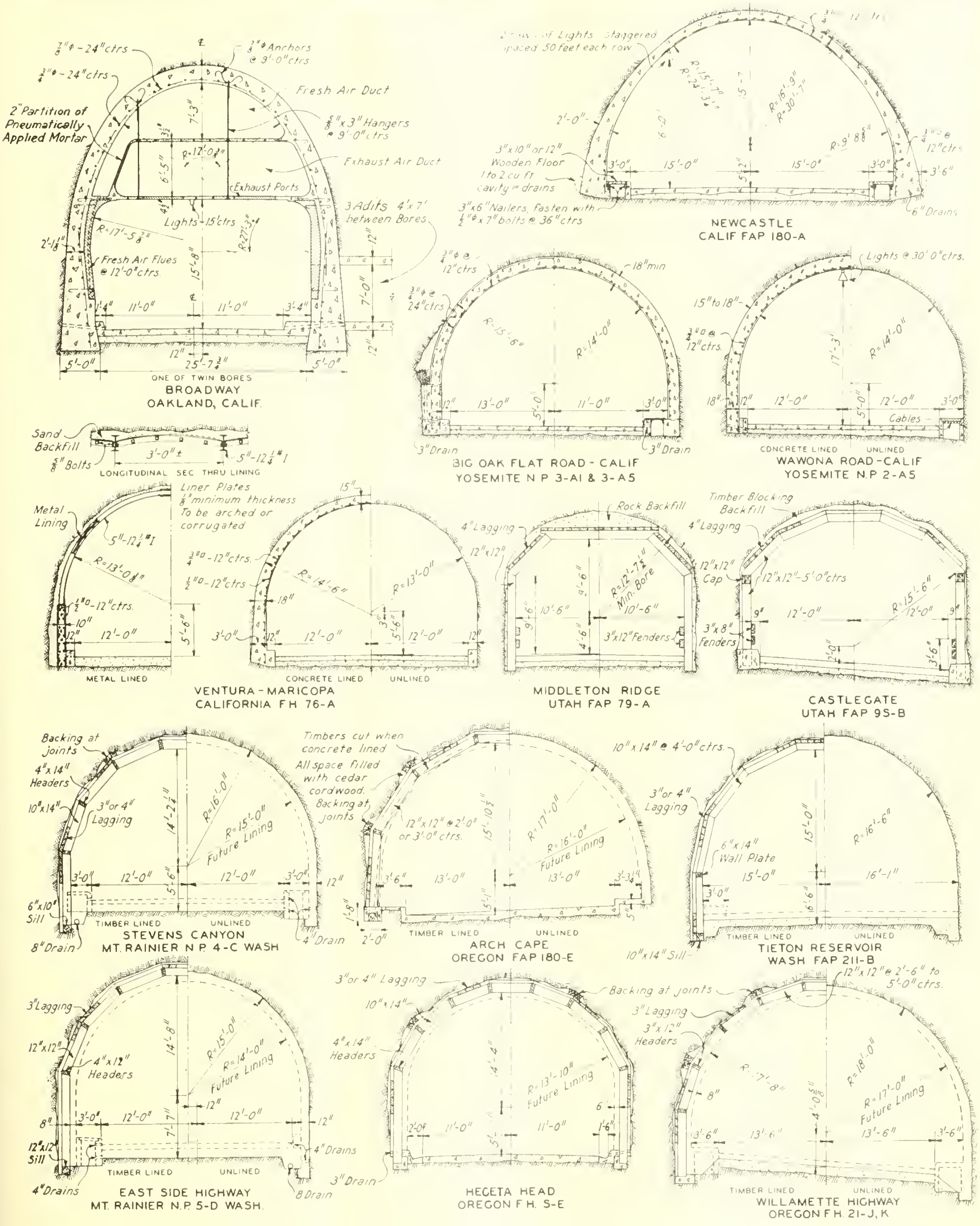

Figure 2.—Cross Sections of Several Highway Tunnels Constructed in the Western States.

TABLE 1.—*Dimensions, costs, and other*

TUNNELS CONSTRUCTED UNDER DIRECTION

Tunnel	Location	State	Length (feet)	Year constructed	Material encountered	Type of lining	Portal structures
Wawona Road, Project 2-A5.	Yosemite National Park.	California	4,233	1932-33	Solid granite and diorite.	Reinforced concrete and pneumatically applied mortar.	One, reinforced concrete.
Big Oak Flat Road, Project 3-A1.	Yosemite National Park.	California	2 tunnels 187 and 353, total 540.	1936-38	Granite (solid and altered).	Reinforced concrete	Four, cement rubble masonry.
Big Oak Flat Road, Project 3-A5.	Yosemite National Park.	California	2,167	1936-38	Granite	Reinforced concrete	None to date
Ventura-Maricopa, forest highway, Project 76.	North of Ventura	California	3 tunnels 170, 128, and 206, total 504.	1931	Shattered shale and serpentine.	Reinforced concrete and steel plate.	Five, reinforced concrete.
Burns Summit, forest highway, Project 7-D.	East of Couer d' Alene.	Idaho	394	1931-32	Talc, shale	Reinforced concrete	Two, reinforced concrete.
Shoshone Reservoir, forest highway, Project 1-B.	West of Cody	Wyoming	4 tunnels 165, 132, 30, and 15, total 342.	1926		None	None
Trans-Mountain Highway, Glacier National Park, Project 1-C.	Glacier National Park.	Montana	187	1926-27	Limestone	None	None
Cave Rock, forest highway, Project 3-D.	East shore of Lake Tahoe.	Nevada	151	1931	Granite	None	None
Heceta Head, forest highway, Project 5-E.	Coast highway	Oregon	680	1930	Clay, boulders, rock	Timber	
Toothrock, Forest Highway Project 28-A.	Columbia River highway.	Oregon	828	1936-37	Basalt	Reinforced concrete	Reinforced concrete
Willamette Highway, Forest Highway Project 21, JK.	Southeast of Eugene.	Oregon	875	1937-38	Granite	Portions with reinforced concrete.	None to date
Mt. Rainier National Park, Project 5-D.	Eastside highway	Washington	510	1937-38	Shattered rock	Timber	
Mt. Rainier National Park, Project 4-C.	Stevens Pass route	Washington	285	1937 38	Solid rock	None	
Zion National Park, Project 1-A2.	East Rim road	Utah	5,613	1927-36	Sandstone	Concrete, pneumatically applied mortar.	One, cement rubble masonry.
Zion National Park, Project 1-A3.	East Rim road	Utah	488	1927-36	Sandstone	Gunite	None

TUNNELS CONSTRUCTED UNDER DIRECTION

Tunnel	Location	State	Length (feet)	Year constructed	Material encountered	Type of lining	Portal structures
Superior-Miami, Federal Aid Project 16.	Superior - Miami highway.	Arizona	277	1919-22	Granite	None	None
Newcastle, Federal Aid Project 180-A.	Newcastle	California	531	1931-32	Granite	Reinforced concrete	Reinforced concrete
Waldo, State Project IV-Marin-1D.	North approach to Golden Gate Bridge.	California	1,000	1937	Shattered shale	Reinforced concrete	Reinforced concrete
Arch Cape, Federal Aid Project 180-E.	South of Seaside	Oregon	1,278	1936-37	Sandstone, shale	Timber	
Cooks-Underwood, Federal Aid Project 112-F.	East of Cooks	Washington	261-No. 1 212 No. 2	1936	Lava	Reinforced concrete	Reinforced concrete[1]
Cooks-Underwood, Federal Aid Project 112-H.	East of Cooks	Washington	130 408 257	1936-37	Lava	Reinforced concrete	Reinforced concrete
Tieton Reservoir, Federal Aid Project 211-B.	West of Yakima, White Pass.	Washington	615	1936-37		Timber for 50-foot each end.	Cement rubble masonry.
Castlegate. Federal Aid Project 95-B.	Near Castlegate	Utah	410	1931	Shale, some coal	Timber	Reinforced concrete
Middleton Ridge, Federal Aid Project 79-A.	East of St. George	Utah	237	1926-27	Sandstone	Timber	Reinforced concrete

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[1] Includes cost of curbs—$3 per foot.

data on western highway tunnels

OF BUREAU OF PUBLIC ROADS

Cost per portal (dollars)	Dimensions of tunnel section	Total tunnel cost (dollars)	Costs per foot for—			Unit costs for principal construction items	Remarks
			Unlined tunnel (dollars)	Lining (dollars)	Lined tunnel (dollars)		
	24-foot roadway, 19-foot center height, 1 3-foot walk.	528,200			125	Driving main tunnel, $80 per lineal foot. Reinforced concrete lining, $50 per lineal foot. Pneumatically applied mortar lining, $30 per cubic yard.	Tunnel costs include portal costs and adit costs, also lighting and ventilation equipment installation; pavement cost excluded.
2,775	24-foot roadway, 19-foot center height, 1 3-foot walk.	129,000	77 and 102	145	240	Driving pioneer bore, $18.41 per lineal foot. Enlarging to size, $66.03 per lineal foot. Concrete lining, $21 per cubic yard. Cement rubble masonry, $34.49 per cubic yard.	Tunnel costs exclude portal and paving costs.
	24-foot roadway, 19-foot center height, 1 3-foot walk.	257,500	119			Tunnel driving, $108 per lineal foot. Adit driving, $20 per lineal foot.	Tunnel costs include: Temporary timbering, pneumatically applied mortar lining, 6- by 7- by 107-foot adit.
1,570	24-foot roadway, 18-foot 6-inch center height, no walks.	62,630	67	*Reinforced concrete* 95 *Steel plate* 46	 165 116	Lined tunnel, $70 per lineal foot. Unlined tunnel, $64 per lineal foot. Reinforced concrete lining, $95 per lineal foot. Steel plate lining, $46 per lineal foot.	Tunnel costs exclude portal costs.
2,196	20-foot roadway, 17-foot center height, no walks.	97,000			245	Tunnel excavation, $60 per lineal foot. Reinforced concrete lining, $106.90 per lineal foot. Reinforcing steel, $0.06 per pound. Concrete, $30 per cubic yard.	Tunnel costs exclude portal costs.
	19-foot roadway, 16-foot center height, no walks.	8,208	24			Enlarging to size, $24 per lineal foot	Cost of enlarging only from 10- by 12-foot tunnels to 19- by 16-foot size.
	19-foot roadway, 18-foot center height, no walks.	22,515	120			Tunnel excavation, $6 per cubic yard	
	24-foot roadway, 18-foot 6-inch center height, no walks.	13,600	90			Unlined tunnel, $90 per lineal foot	Concrete curbs not included in tunnel cost.
	22-foot roadway plus 2 1-foot 6-inch gutters, 20-foot center height, no walks.	60,700			88	Tunnel excavation, $66.50 per lineal foot. Timber, $55 per thousand feet board measure.	Curbs not included.
6,400	26-foot roadway, 20-foot center height, 2 4-foot walks.	177,385			214	Tunnel excavation, $85 per lineal foot. Concrete, $22.50 per cubic yard. Timber, $55 per thousand feet, board measure. Cement rubble masonry, $12 per cubic yard.	Tunnel cost includes lighting system installation for $6,680.
	26-foot roadway, 21-foot center height, 2 3-foot 6-inch walks.	91,475	80		152	Unlined tunnel, $80 per lineal foot. Lined tunnel, $100 per lineal foot. Timber, $65 per thousand feet, board measure.	300-foot reinforced concrete lining 575-feet unlined.
	24-foot roadway 21± -foot center height, 1 3-foot walk.	69,400			136	Tunnel excavation, $89 per lineal foot. Timber, $60 per thousand feet, board measure.	Change order during construction will modify costs considerably.
	24-foot roadway, 20± -foot center height, 1 3-foot walk.	26,180	92			Tunnel excavation, $90 per lineal foot	Timbering omitted.
4,000	20-foot roadway, 16-foot center height, no walks.	790,628			140	Driving tunnel, $55 per lineal foot. Pneumatically applied mortar, $24 to $26 per cubic yard. Concrete lining, $50 per lineal foot. Reinforcing steel, $0.04 to $0.05 per pound.	Costs include galleries. Costs not segregated between lined and unlined portions.
	20-foot roadway, 16-foot center height, no walks.	31,130			64	Driving tunnel, $55 per lineal foot. Pneumatically applied mortar, $26 per cubic yard.	Lined with pneumatically applied mortar only.

OF STATE HIGHWAY DEPARTMENTS

Cost per portal (dollars)	Dimensions of tunnel section	Total tunnel cost (dollars)	Unlined tunnel (dollars)	Lining (dollars)	Lined tunnel (dollars)	Unit costs for principal construction items	Remarks
	2 lanes, no walks	16,032	58			Tunnel excavation, $6 per cubic yard	
3,500	30-foot roadway, 20-foot 9-inch center height, 2 3-foot walks.	126,300			238	Tunnel excavation, $120 per lineal foot. Reinforced concrete lining, $110 per lineal foot.	Tunnel cost includes installation of electric lighting system for $1,500.
7,500	42-foot roadway, 28-foot 9-inch center height, 1 3-foot 4-inch walk.	483,500	225	258	483	Tunnel excavation, $4.40 per cubic yard. Reinforced concrete lining, $220 per lineal foot. Reinforced concrete lining, $165 per lineal foot. Reinforcing steel, $0.055 per pound.	Installation of lighting system, $2,160.
	26-foot roadway, 21-foot center height, 2 3-foot 6-inch walks.	200,138			157	Tunnel excavation, $3.50 per cubic yard. Timber, $70 per thousand feet, board measure.	Future reinforced concrete lining.
3,110-No. 1 870-No. 2	24-foot roadway, 20+foot center height, no walks.	No. 1-56,982 No. 2-27,270			218 129	Tunnel excavation, $4 per cubic yard. Concrete, $16 and $18 per cubic yard. Reinforcing steel, $0.05 per pound.	High cost of tunnel No. 1 due to cave-in on adjacent railroad tunnel caused by highway tunnel construction.
1,160 1,535 1,770	24-foot roadway, 20+foot center height, no walks.	15,632 52,645 43,837			120 124 171	Tunnel excavation, $3.88 per cubic yard. Concrete $16.50 per cubic yard. Reinforcing steel, $0.05 per pound.	High cost of tunnel lining in No. 3 due to increased thickness of lining.
1,850	24-foot roadway, 21-foot center height, 1 3-foot walk.	65,600	114	38	162	Tunnel excavation, $4 per cubic yard. Timber, $80 per thousand feet, board measure. Cement rubble masonry, $18 per cubic yard.	
1,815	24-foot roadway, 17-foot 6-inch center height, no walks.	38,400			95	Tunnel excavation, $60 per lineal foot. Timber lining, $60 per thousand feet, board measure. Concrete, $18 per cubic yard plus cement at $0.82 per sack.	
1,800	20-foot roadway, 14-foot center height, no walks.	20,400			86	Tunnel excavation, $48 per lineal foot. Concrete, $27.50 per cubic yard plus cement at $0.83 per sack. Timber, $110 per thousand feet, board measure.	

TABLE 1.—*Dimensions, costs, and oth*

TUNNELS CONSTRUCTED UNDE

Tunnel	Location	State	Length (feet)	Year constructed	Material encountered	Type of lining	Portal strutures
Broadway Tunnel	Oakland	California	Twin bores 3,203 and 3,135.	1934–37	Shattered shale	Reinforced concrete	Monumental reinforced concrete.
Yerba Buena Island	San Francisco-Oakland, Bay Bridge.	California	540	1936–37	Shattered shale	Reinforced concrete	Monumental reinforced concrete.
Figueroa Street Extension	Los Angeles	California	3 tunnels 46, 130, 105	1936		Reinforced concrete	Reinforced concrete

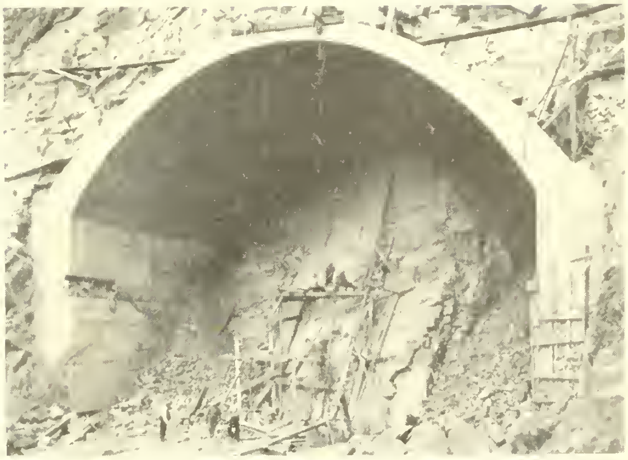

Photo by the California Toll Bridge Authority.

FIGURE 3.—THE YERBA BUENA ISLAND TUNNEL, CALIFORNIA, ILLUSTRATING THE GERMAN METHOD OF TUNNELING.

For short tunnels on tangents the brief time required to pass through the tunnel and the infiltration of light from the portals remove the necessity for lighting at the portals.

Where tunnels 1,500 feet or longer are required, or where vertical curves which might create gas pockets are encountered, provision must be made for ventilation. It has been the policy of the Bureau of Public Roads to consult with the United States Bureau of Mines on ventilation problems in the design of the major tunnels it constructs. Natural ventilation, from differences in barometric pressure, is generally unreliable. However, carbon monoxide, which is the principal gas that must be removed from tunnels, is lighter than air, and when heated is much lighter than cool air. It therefore tends to flow upward along the top of the tunnel, especially if the location is on a continuous grade.

Under usual conditions a composite[3] car will exhaust carbon monoxide on grades of 4 percent or less at a rate of 1.5 cubic feet per minute per car. Assuming 2 cubic feet per minute and assuming 2 parts in 10,000 to be the maximum concentration allowable, provision must be made for supplying 5,000 cubic feet of fresh air per minute per car. Using "transverse" ventilation on tunnels over 3,000 feet long, considerable tunne space is required for the large air ducts needed to provid for the peak traffic. In transverse ventilation the fresh air is blown in through ducts having ports generally at the same elevation as the seats of passenge cars, spaced at regular intervals along the walls, and exhausted through a crown duct. This system is no widely used in tunnels in rural locations. The transverse system is not, of course, adapted to use in unlined tunnels.

In the "longitudinal" ventilation system the necessity of ducts is eliminated if adits or shafts can be strategically introduced as provided, for example, in the design of the Wawona tunnel in Yosemite Nationa Park. In this ventilation system provision is made to exhaust the contaminated air at a point or points along the bore through shafts or adits, the fresh air entering through the portals or through adits or shafts acting in conjunction with the portals. The longitudinal ventilation system can be used on any length of tunnel provided openings are constructed at intervals not more than ½ mile apart. Under such conditions, ventilation is an important economic factor in respect to both firs cost of installation and operating charges.

A combination of the transverse and longitudina ventilating systems might be economical in tunnels o moderate length where shafts and adits are not feasible Under a combination design, the gas would be exhausted along a crown duct, while the fresh air would come in through the portals, using the roadway area. No additional fresh air duct would be provided as in the case of the transverse ventilation system.

Advocates of the transverse system claim that it is superior in localizing a fire by preventing the smoke and flame from traveling along the bore. The accuracy of this assumption is very doubtful in the case of a major conflagration, since a tunnel, especially one on a grade would probably act as a chimney under either the transverse or longitudinal system. Under the above emergency, after all individuals had been rescued, the fans should be stopped to avoid possible heat damage to the fans and other equipment. Stopping the fans would also help to smother the fire. A better contro probably would be the installation of fire doors.

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[3] The "composite" car used in the design of a tunnel ventilation system should be representative of the traffic expected to use the tunnel. That is, it should produce the same amount of contamination as the average vehicle using the tunnel.

data on western highway tunnels—Continued

DIRECTION OF OTHER AGENCIES

Cost per portal (dollars)	Dimensions of tunnel section	Total tunnel cost (dollars)	Costs per foot for— Unlined tunnel (dollars)	Lining (dollars)	Lined tunnel (dollars)	Unit costs for principal construction items	Remarks
Not segregated.	Each bore 22-foot roadway, 15-foot 8-inch center height, 1 3-foot 4-inch walk.	Original estimate 4,173,000.			1,320	Driving tunnel, $325 per lineal foot. Ventilating, lighting for $785,000.	Costs include driving, lining, paving, etc. Constructed by Joint Highway District No. 13.
Not segregated.	Upper deck, 58-foot roadway, 29-foot center height, 2 3-foot 9-inch walks. Lower deck-truck lane, 31-foot roadway, 16-foot center height. Railroad space, 27-foot horizontal, 20-foot height.	851,453			1,580	Original lump sum bid $610,000 for all items increased by changes to $851,435.	Constructed by California Toll Bridge Authority and State Division of Highways.
No data	40-foot roadway, 28-foot 3-inch center height, 1 5-foot walk.	511,166			543	Tunnel costs include all items of construction, driving, lining, paving, portals, etc.	Constructed by local improvement district.

PERMANENT LINING FREQUENTLY NEEDED IN TUNNELS

In adopting a cross section for tunnels, even in self-supporting rock formations, experience indicates that sufficient area should always be provided for a minimum thickness of lining.

In adopting a cross section for rural tunnels, it must be borne in mind that excavation for a tunnel in rock formation will, in general, cost about $5 per cubic yard. Also, a vertical clearance of 14 feet is desirable. This vertical clearance generally makes it economical to design with a semicircular intrados section with the springing lines from 3 to 5 feet above the pavement grade. This form of section allows space for at least 3-foot sidewalks with no infringement of clearance on traffic lane widths.

At best, lighting conditions at tunnel portals are not ideal and ample roadway is, therefore, essential. Some of the older vehicular tunnels have meager widths. The travelway of the East Rim Road tunnel in Zion Nation Park (see figure 1) is only 20 feet. In newer tunnels the travelways are wider; for example the Wawona tunnel travelway is 24 feet wide; the Tooth Rock tunnel travelway is 26 feet wide; and the Willamette tunnel, which is on a curve of 4°36′, has a travelway 27 feet wide. The Waldo four-lane tunnel has a 42-foot, four-lane travelway and one 3⅓-foot sidewalk. With regard to the East Rim Road tunnel's narrow section, it should be noted that the tunnel is within a National Park and the traffic moves slowly to take advantage of the vistas afforded from the tunnel's several galleries.

In designing timber lining sets, there is generally a choice between 5 to 7 segments (see fig. 4) and the use of a wall plate between the posts and the first segments. Where sets are not to be removed when permanent lining is placed, wall plates should be omitted. In ground that requires driving with side headings at the springing lines (see fig. 5) it is generally easier to keep the segments true to section if side plates are used. These plates are difficult to remove when setting posts as the core is excavated, and should be omitted if possible.

Mud sills are generally needed under the footings of posts, but where posts are notched into rock formations, blocks to give uniform bearing are equally serviceable. Their function is to provide even bearing and prevent kicking in at the bottom. It has been the general practice to design sets with untreated timber, using 12- by 12-inch material for plates, posts, and mud sills.

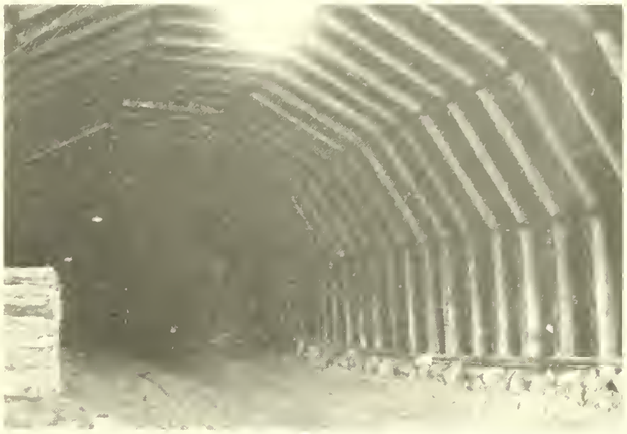


Figure 4.—The Arch Cape Tunnel, Oregon, With Timber Lining in Place. The Lining Sets Consisted of Five Segments, Wall Plates, and Posts.

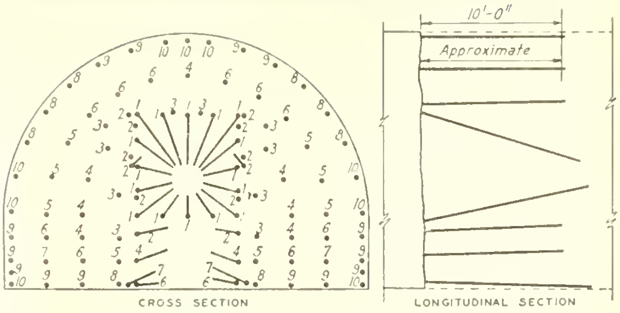
Headers, 4 by 12 inches, with 3- or 4-inch lagging are required.

As a safety measure in heavy ground and as a temporary item during construction, horizontal cross struts or square sets are usually required. (See figs. 6 and 7.)

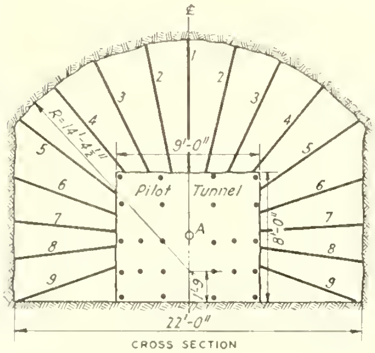
Timber sets designed for the Stevens Canyon-Mount Rainier National Park tunnel (see fig. 2), where the sets were expected to serve for an extended period, consisted of 10- by 14-inch material fastened by 1- by 6-inch dowels at the joints. Longitudinal bracing consisted of 4- by 14-inch headers held with twentypenny spikes. The bottom of the post was set in rock channels on 6- by 10-inch blocks.

In packing the space behind and above the timber lining, (see fig. 8) it is generally the practice in forest areas to use the class of wood available locally. Over permanent lining hand-placed stone is desirable. In large cavities, sand can easily be pumped back of the concrete lining if pipes are placed for this purpose during concreting. (See fig. 9.) Sand can also be placed advantageously behind steel liner plates. Where the rock contains open cracks or faults sand will probably not be satisfactory, but lean concrete can be used to advantage. Sand was used successfully on the East Rim Road tunnel in a large overbreak section, and on the Maricopa-Ventura tunnel behind the liner plates on very thin sections.

In designing tunnel linings, considerable judgment



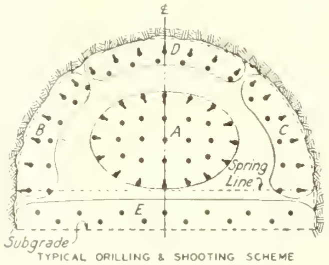
DRILLING DIAGRAM TUNNEL NO. 3
BIG OAK FLAT ROAD



Notes on Pilot Tunnel drill holes.
Top row holes = Back holes.
First row down from top row = Breast holes.
Second and third row down from top row = Cut holes and easers.
Bottom row = Lifter holes.

Notes on Full Bore Section.
Numbered heavy lines indicate ring drill holes and directions from "A" drilled after Pilot Tunnel is excavated.
"A" indicates position of Spindle on which Stoper Drill used for ring drilling is revolved. "A" is 4 feet above profile grade on ℄ of tunnel. Spindle is attached to iron column anchored between the floor and roof of Pilot Tunnel.

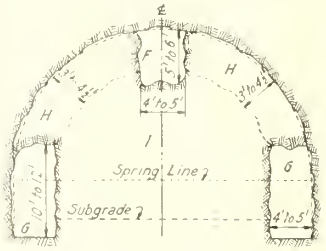
ZION N.P. UTAH
I-A2 & I-A3



TYPICAL DRILLING & SHOOTING SCHEME
EAST FACE OF TUNNEL

Notes:-
1. Number of holes 55 to 90 depending on material encountered
2. Depth of holes 4 to 8 feet.
3. Explosives 1 to 1¼ pounds per Cu. Yd.

Order of Shooting
1. Group A
2. Group B & C
3. Group D
4. Group E



TYPICAL METHOD OF WORKING
WEST FACE OF TUNNEL

Note -
F - Crown Drift or Heading
G - Side Drifts or Heading
H - Peripheral Ring
I - Core.

Method of Excavation.
1. Sides and Crown drifts extended into face 10 to 20 feet.
2. Peripheral ring progressively stoped out between drifts to permit placing of timber lining.
3. Center core removed after timber lining in place

TOOTH ROCK
OREGON F.H. 28-A

Figure 5.—Drilling Plans Followed in the Construction of Three Western Highway Tunnels.

must necessarily enter into the determination of crown depth. In earth formations, the depth of active overburden can be estimated with reasonable accuracy. Having determined the maximum depth of active overburden, the necessary thickness of lining can be calculated by applying the formulas used in designing other arch structures. E. Lauchli's work on tunneling gives a method of determining the thickness of lining based on measurements of crown deflection. Deflections at the crown in most rock formations would be so small that other considerations, in general, determine the minimum thickness of lining. In earth formations, it is best to make a very liberal allowance for the weight of the overburden to be carried by the lining. The theory of earth pressure deals with dry granular masses under ideal conditions, but where saturated soil is encountered, indeterminable conditions develop and considerable temporary timber lining becomes necessary before permanent lining can be placed. Under such conditions the performance of the temporary timber lining is generally a satisfactory guide in designing a permanent concrete or steel lining.

Generally, tunnels are needed only in rock formations, since mountains with soil cover are usually of such easy slopes as to make tunneling unnecessary. Rock masses are generally self-supporting and lining is only required to support shattered areas or to protect sections through fault planes.

Judgment based on experience with various geological formations must be used in estimating the maximum depth of rock fragment which might become detached and hence need support. Generally, 5 feet would be an ample depth, and if the lining is placed before fragments become loosened, there is quite a factor of safety by reason of the internal friction of the fragments before movement can begin. Calculations will ordinarily indicate a depth of lining much less than the minimum that can be efficiently placed by the best methods. Using concrete, a crown thickness of 12 inches is about the least that can be successfully placed in tunnel linings. A composite lining of steel plates, I-bars, and concrete, allows a very shallow depth of lining with a minimum of 8 inches at the crown.

Considerable portions of the Wawona and East Rim Road tunnels were entirely self-supporting, and a minimum thickness of pneumatically applied mortar was used to prevent raveling and weathering on some sections.

The East Rim Road tunnel was originally lined with pneumatically applied mortar over the self-supporting sections and timber was used on portions where needed. (See fig. 1.) Later this timber showed considerable distress due to the working of the cliff faces, and steel members were installed in the sections having the greatest movement. Ten-inch, 21-pound I-beams spaced 3 feet between centers and set in concrete have been sufficient to prevent any further movement.

PORTALS SHOULD HARMONIZE WITH SURROUNDINGS

The extensive use of pneumatically applied mortar in the Wawona tunnel produced a very pleasing finish. (See fig. 10.) This treatment softened the appearance of angular breakage of the rock.

Pneumatically applied mortar is especially adapted to irregular sections where form building would be difficult. In one gallery of the East Rim Road tunnel, application of the mortar solved a very troublesome problem. The roof of this gallery, which was very irregular in contour over its 100-foot span, gave indication, when sounded, that cracks were developing which might later allow sizeable fragments to fall. If an attempt had been made to fit lining forms, considerable dangerous and expensive trimming would have been necessary, and the long spans would have made stress determinations very difficult. The plan adopted was to grout 1-inch anchor rods to a depth of 9 feet. These rods were spaced 5 feet between centers, and held by hooked ends a grillage of reinforcement made up of ½-inch rods spaced 12 inches between centers. This grillage was securely imbedded in a 6-inch slab of mortar placed by the pneumatic method. It has solved the problem economically and presents a very agreeable appearance.

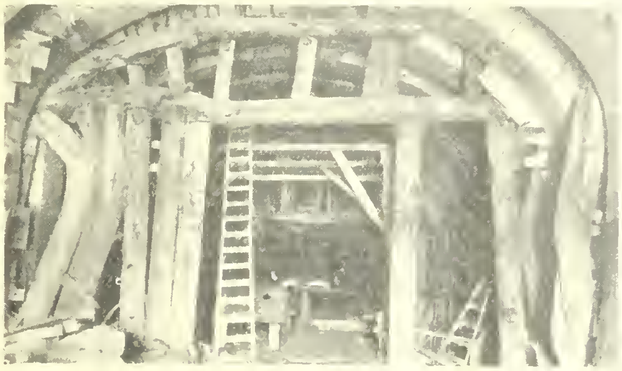


Figure 6.—Square Sets that Failed Because of Heavy Ground Encountered, in the North Pacific Forest Highway Tunnel, Idaho.

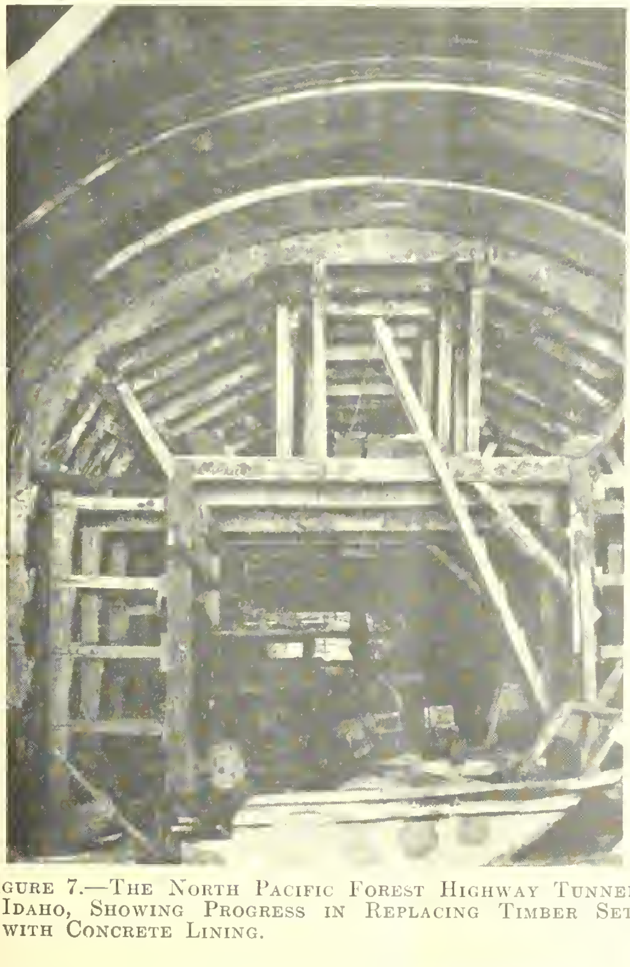


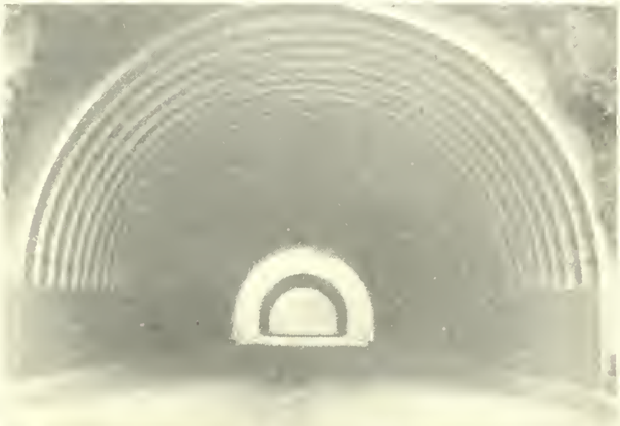
Figure 7.—The North Pacific Forest Highway Tunnel, Idaho, Showing Progress in Replacing Timber Sets with Concrete Lining.

In wet formations provision for tunnel drainage is necessary. Where saturated soil cover is encountered, tile should be used to convey the water through to the portals. Grouting rock seams is generally not satisfactory, but where a flow of water is present, tile should be laid behind the tunnel lining.

Tile should also be laid in the subgrade beneath the road surface. The area below the sidewalks is a convenient location for drainage and wiring facilities.

If water would tend to flow on the pavement into the portals, surface grillage drains should be located outside the portals, tranverse to the tunnel axis, and complete for the entire width of roadway.

Natural rock slopes at tunnel portals give a pleasing appearance, but some protective work is generally necessary, if portal structures are not used, to prevent drip and icicle formation which may become a traffic hazard.



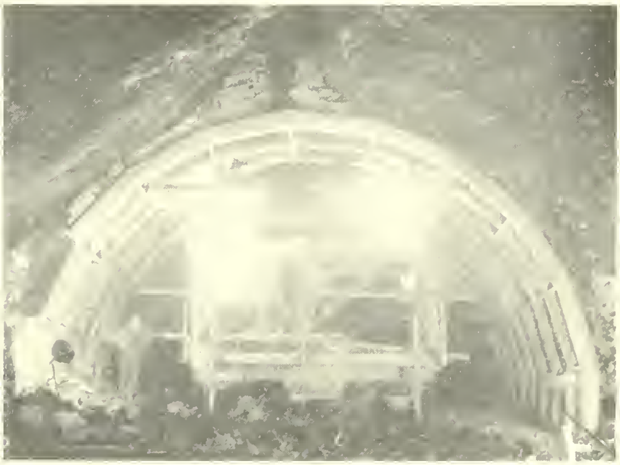


Figure 8.—Upper, Steel Plate Lining in the Maricopa-Ventura Tunnel, California. Lower, Tooth Rock Tunnel, Oregon, Showing a Hole Outside the Neat Tunnel Line Being Filled with Wood.

In tunnels over 1,500 feet long, where the construction of galleries is impractical, some form of mechanical ventilation is necessary. In rural tunnels, the longitudinal system of ventilation will generally be found to be more economical. The fans should be placed far enough away from the roadway to be clear of any flame that might develop from burning oil tank wrecks. The exhaust adits or shafts should be so located as to draw gas from the upper area of the bore.

The ventilation machinery should operate automatically under normal conditions, and there should also be manual controls. It should also be capable of reversing the direction of air currents, as is possible in the Wawona tunnel.

The control room should not be within the tunnel, but should be located at a point where it will be quickly accessible in case of a fire within the tunnel. In case of a bad fire, the ventilation system could not be expected to handle all of the escaping smoke indefinitely,

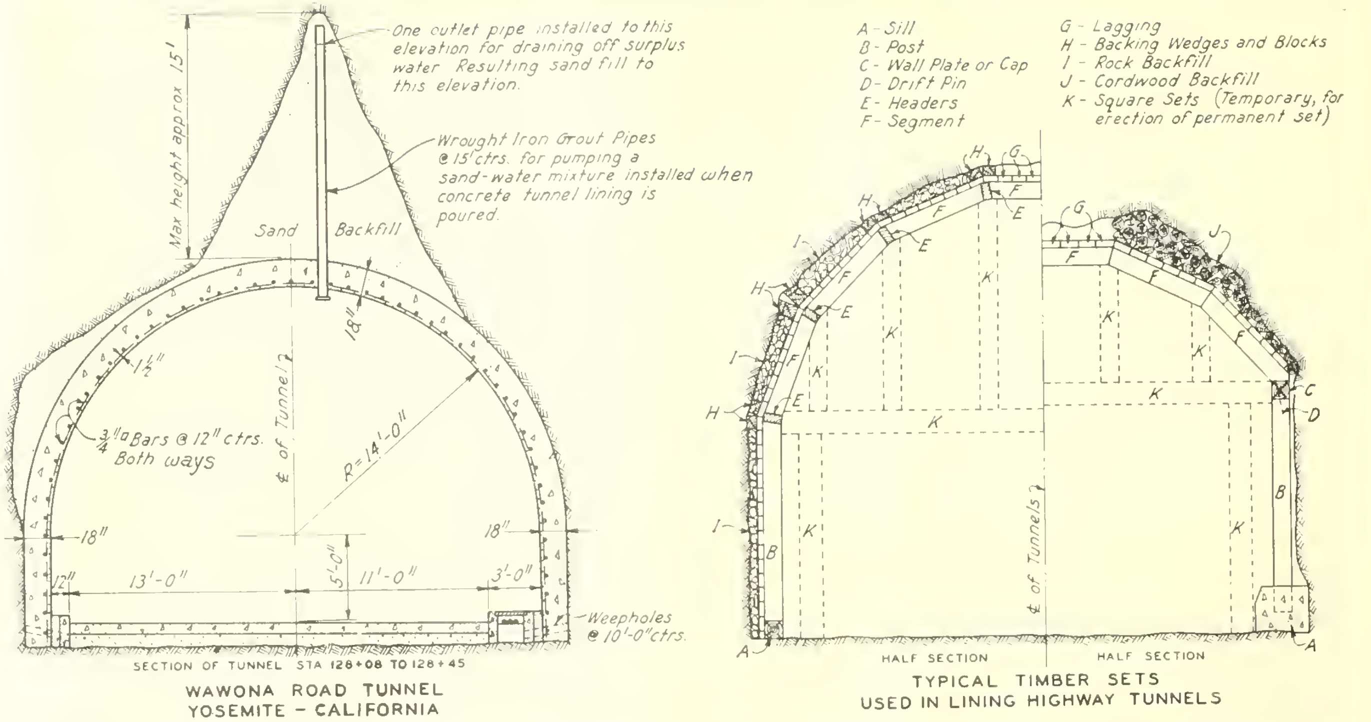

One outlet pipe installed to this elevation for draining off surplus water Resulting sand fill to this elevation.
Max height approx 15'
Wrought Iron Grout Pipes @ 15' ctrs. for pumping a sand-water mixture installed when concrete tunnel lining is poured.
Sand Backfill
18"
1½"
¾"⌀ Bars @ 12" ctrs. Both ways
℄ of Tunnel
R=14'-0"
18"
18"
12"
13'-0"
5'-0"
11'-0"
3'-0"
Weepholes @ 10'-0" ctrs.
SECTION OF TUNNEL STA 128+08 TO 128+45
WAWONA ROAD TUNNEL
YOSEMITE - CALIFORNIA
A - Sill
B - Post
C - Wall Plate or Cap
D - Drift Pin
E - Headers
F - Segment
G - Lagging
H - Backing Wedges and Blocks
I - Rock Backfill
J - Cordwood Backfill
K - Square Sets (Temporary, for erection of permanent set)
A
B
C
D
E
F
G
H
I
J
K
℄ of Tunnels
HALF SECTION
HALF SECTION
TYPICAL TIMBER SETS
USED IN LINING HIGHWAY TUNNELS

Figure 9.—Left; Cross Section of Wawona Tunnel, California, Showing Sand Backfill Above Concrete Lining. Right; Typical Timber Sets Used in Lining Highway Tunnels.

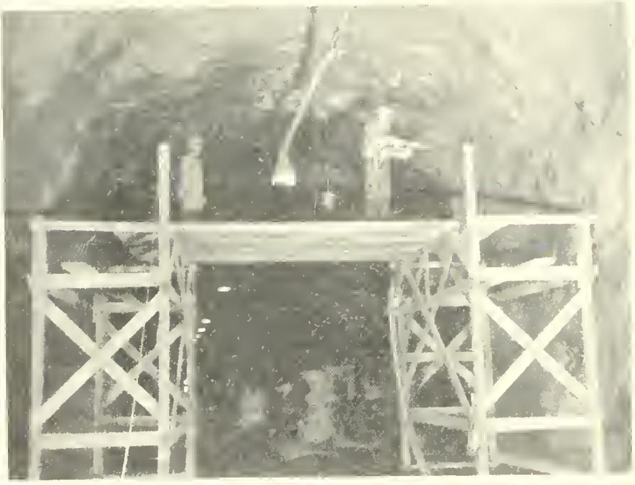


Figure 10.—Pneumatically Applied Mortar Being Placed in the Wawona Tunnel, California.

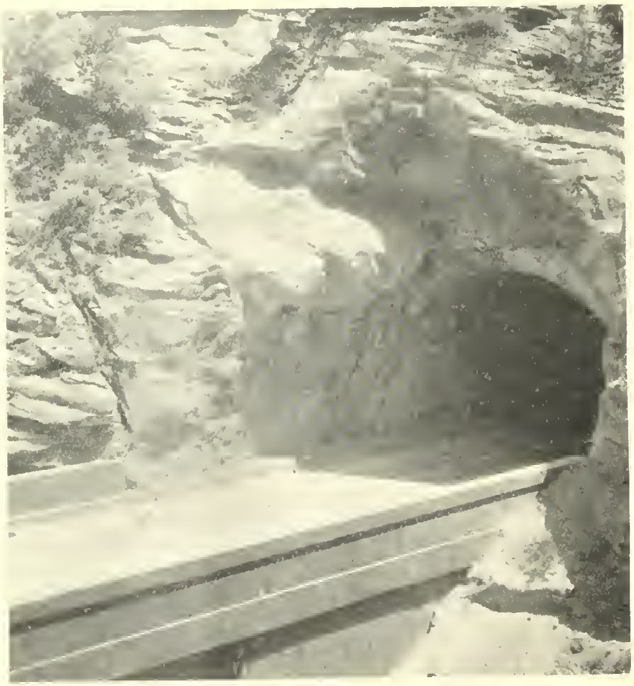


Figure 11.—East Portal of the East Rim Road Tunnel, Utah. Strong, Durable, Native Rock at this Portal Made Construction of a Portal Structure Unnecessary.

but only long enough to permit all travelers to escape from the tunnel.

The principal function of a tunnel portal is to support the weaker strata generally found at the ground surface. Portals generally reduce the length of the bore and the lining. From a landscape viewpoint they are not as attractive as natural formations (see fig. 11).

The portal design should be in keeping with the surrounding formations. The design of the lower portal to Maricopa-Ventura tunnel No. 3, while attractive (see fig. 12), could have been improved by introducing coloring material into the concrete mixture to darken the finish to the shade of the surrounding formations. The lining ring should be a shade lighter than the concrete work above the extrados.

The west portal of the Wawona tunnel (fig. 13) illustrates one method of portal treatment. The smooth portal harmonizes with the regular, even rock slopes above the parapet. The curved extrados at the left accentuates the arched section of the tunnel, and gives the appearance of stability to the portal in sympathy with the formation of the adjacent cliff. This unobtrusiveness in line may be compared with the jagged appearance of the Big Oak Flat tunnel portal (fig. 14) where the masonry lines give much the broken effect of the shattered rock formations of the cover. The curved line of the parapet also carries out the scheme of the tunnel arch.

The Tooth Rock tunnel portals have very pleasing appearances as may be observed in figure 15 and in the cover illustration. The arch of this tunnel is carried through in the curved upper line of the parapet. The long curve of the wing walls blends into the retaining walls. The slopes above the portal structure are to be stabilized and improved by appropriate planting of local shrubs. The broken lines of the masonry above the arch ring give the effect of informality.

In contrast to these rural tunnel portals are the monumental structures of the Yerba Buena tunnel of the San Francisco-Oakland Bay Bridge. (See fig. 16). It was fitting that the portals of this tunnel be in keeping with the entire project. They are not too much dwarfed by the adjacent towers of the spans, and the smooth concrete harmonizes with the longer lines of other parts of the project.

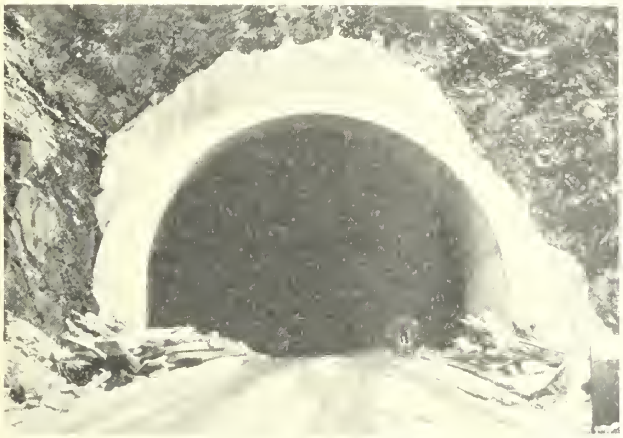


Figure 12.—Portal Structure of the Maricopa-Ventura Tunnel, California. The Structure Was Needed to Reinforce the Badly Shattered Native Rock.

In the future more attention will undoubtedly be paid to the lighting of highway tunnels, especially at the portals during the brighter portion of the day. Good lighting is necessary as a safety measure because of the slowness with which the eye adapts itself to a change from bright light to darkness. Night lighting of tunnels is primarily for the benefit of pedestrians and need be considered only near population centers.

The lighting system of the Broadway tunnel between Alameda and Contra Costa Counties, Calif., makes use of louver frames (see fig. 17) which gradually diffuse the light over a distance of 200 feet at the approaches of the portals. Truss frames, supported on concrete side walls, span the roadway. The trusses are spaced at approximately 34-foot centers and from their lower chords are suspended four lines of purlins consisting of 12-inch, wide-flange, 22-pound I-beams. These purlins support a series of grillages. The grillages consist of a series of vertical and oblique aluminum vanes so oriented that during no season of the year can the sun's rays pass directly through to the roadway. Openings were made through the louver panels, and lighting units have been installed.

LIGHTING INSTALLATION IN TOOTH ROCK TUNNEL DESCRIBED

The system serves the purpose very satisfactorily in the mild climate of this locality, but in a colder climate, provision to handle snow would be necessary. One possible means of eliminating snow would be a shed covering with sufficient pitch for gravity snow disposal. The roof could be made as transparent as desirable by using glass panels.

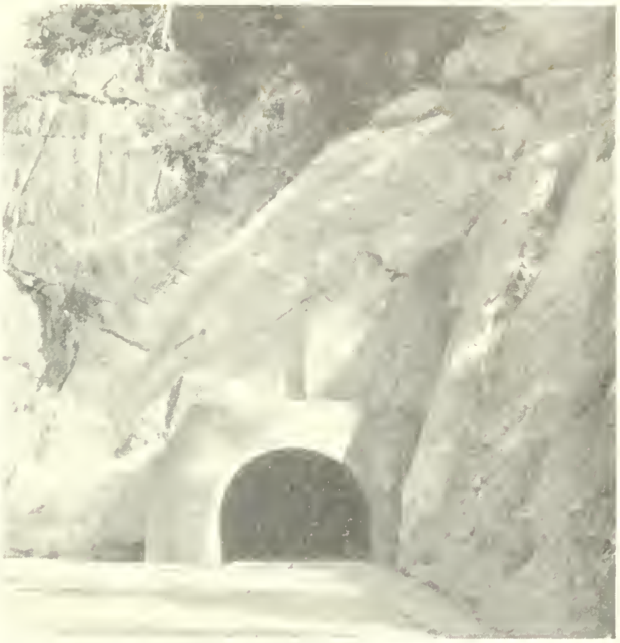


Figure 13.—West Portal of the Wawona Tunnel, California. The Smooth Concrete of the Portal Harmonizes with the Adjacent Rock Faces.

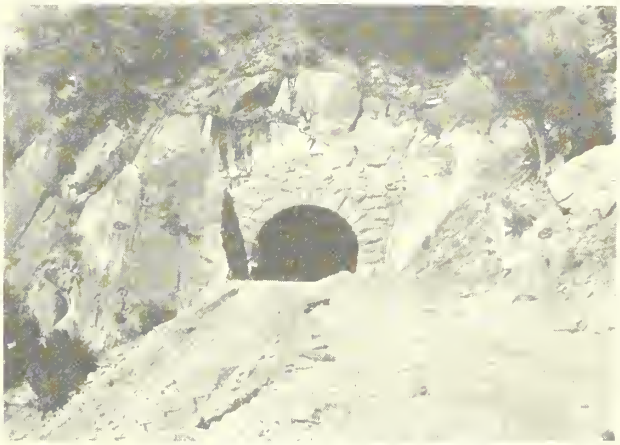


Figure 14.—Portal of the Big Oak Flat Tunnel, California. The Masonry Portal Harmonizes with the Shattered Rock Around It.

The Tooth Rock tunnel in Oregon is the only rural highway tunnel in the west equipped with portal daylight illumination, and is of particular interest.[4]

The illumination within the tunnel is of sufficient intensity to provide for safe travel at normal speeds in the most adverse conditions of bright sunlight. In the past, a serious fault of highway tunnels has been the hazard to traffic created by driving from bright daylight into darkness.

Before the Tooth Rock tunnel was constructed, the Bureau of Public Roads cooperated with the State of Oregon in making comparative field tests of lighting in existing tunnels. These tests covered overhead

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[4] This tunnel is described by H. D. Farmer, Senior Highway Engineer, United States Bureau of Public Roads.

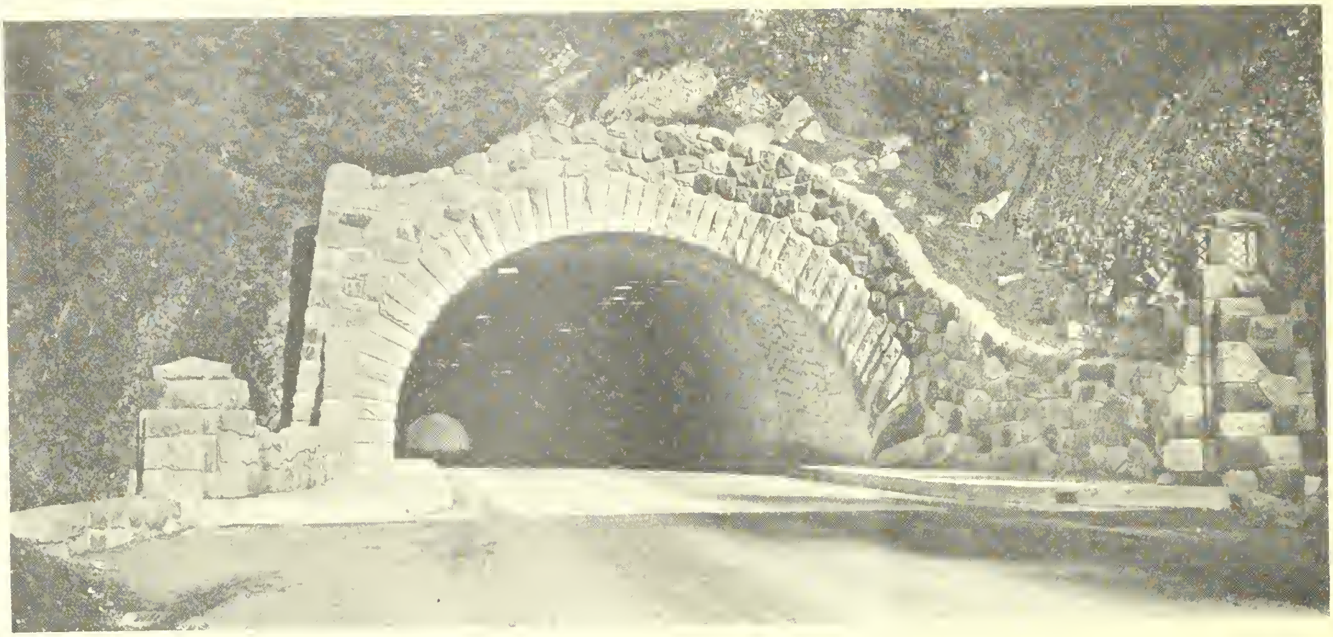

Figure 15.—West Portal of Tooth Rock Highway Tunnel, Oregon.

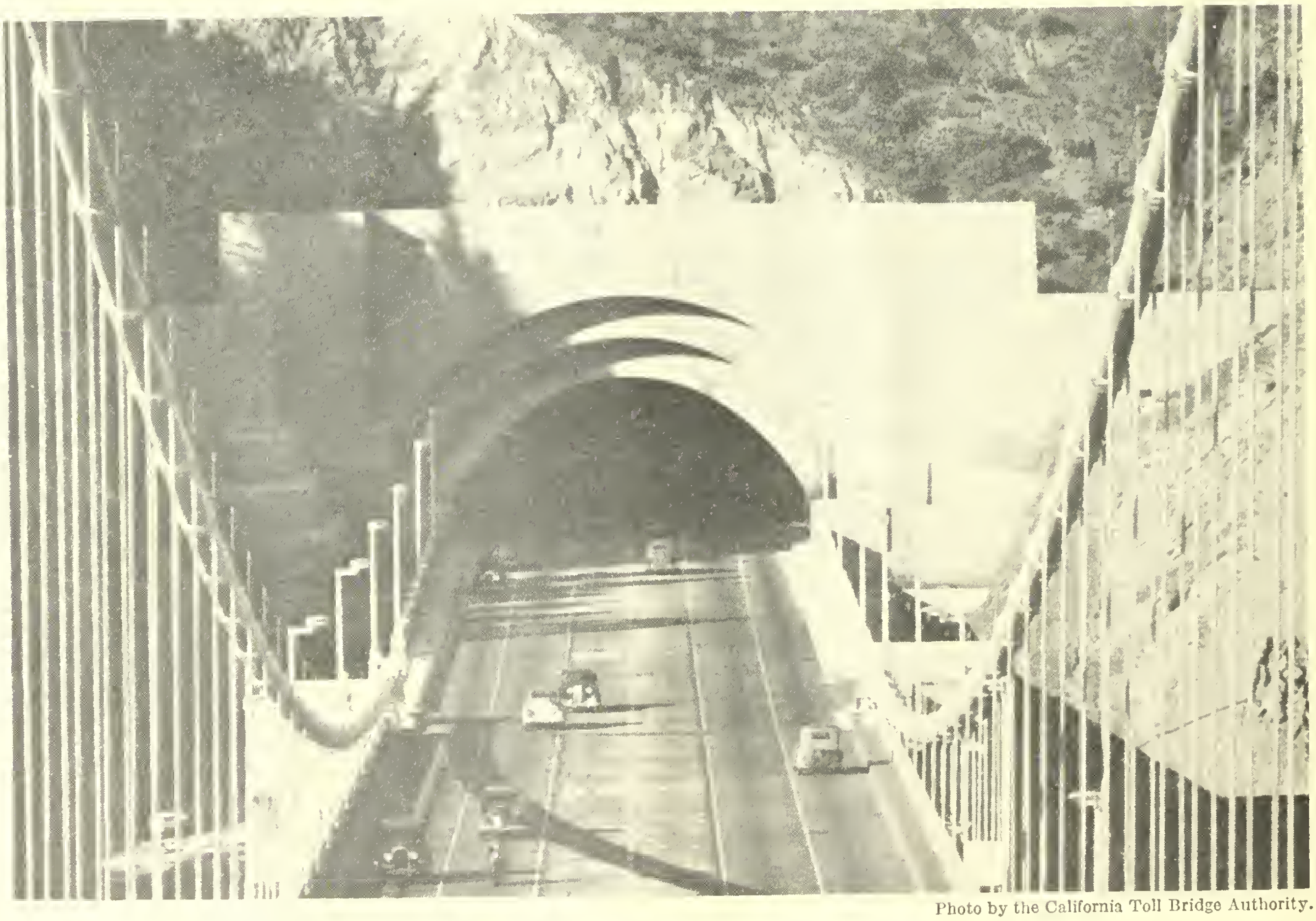

Photo by the California Toll Bridge Authority.

Figure 16.—Upper Deck of the Yerba Buena Island Tunnel, California.

incandescent and sodium lights with low-height, side-mounted fixtures with controlled beams. A system of overhead lighting was found to be more effective than side lights. The overhead lights provided a full, normal silhouette of vehicles that are passed in the tunnel, whereas the side lights illuminated the chassis only and the result was unsatisfactory for vehicles passing at normal speeds.

The installation was designed to provide a transition from bright daylight at the portals (often in summer from 10,000 to 15,000 foot-candles) to a suitable intensity of illumination within the tunnel. An intensity of 6 foot-candles at the road surface was found to be satisfactory. The majority of the lighting units used were sodium vapor luminaires. The greater economy of operation of sodium vapor lamps as compared with incandescent lamps of equal capacity was the controlling factor in their adoption. However, near the portals, incandescent lights were used in conjunction with sodium vapor fixtures to build up a high intensity with fewer units. In this tunnel each 1,500-watt incandescent lamp produces about 33,000 lumens while the largest sodium unit produces 10,000 lumens.

There are in all 20 sodium vapor reflector luminaires of the concealed source type designed to give adequate and uniform pavement brightness and freedom from objectionable glare.

The units have the following characteristics: Straight series, type NA-9, 10,000 lumens, 31.4 volts, 6.6 amps. and 195 watts. The reflectors and units are similar to the ones originally designed for lighting traffic circles, and are manufactured from specular aluminum.

Each unit is mounted horizontally with the major axis transverse to the roadway, and 18 feet above the pavement. The close spacing of the sodium vapor lamps results in a uniform intensity of 6 foot-candles under the fixture at the roadbed. For convenience in relamping the luminaires a special design was worked out so that the units can be swung free of the concrete lining. Plug type absolute cut-outs have been provided in a flush mounted box adjacent to the sodium luminaire for disconnecting this unit from the high voltage series circuit.

All sodium luminaires are operated in a single series circuit (see fig. 18), part of which is short circuited during the hours of darkness by means of a remote-control oil switch controller. A second similar controller is provided on the primary side of the regulating outdoor transformer for deenergizing the entire circuit by means of a low-voltage switch. This controller is normally open and can close only when its operating coil is energized; this provision is for safety.

A comparison of power consumption shows that sodium vapor lamps rated at 10,000 lumens consume approximately 195 watts per hour against 526 watts per hour for similar incandescent lamps.

Nine incandescent units (see fig. 19), each consisting of seven 1,500-watt lamps and two 1,000-watt lamps furnish approximately 270,000 lumens and provide an effective transition from bright sunlight to the dimness of the tunnel. These incandescent lamp units are grouped as shown in figure 18 within a small area on the incoming traffic side of the tunnel.

Fixtures for the incandescent lights are standard reflectors mounted 16 feet above the roadway surface. The incandescent lights are connected by three-wire multiple connections in two separate circuits supplied independently by two distributor transformers located in vaults at their respective ends of the tunnel. Control is effected through remote-control, low-potential contactors.

The incandescent units with their concealed-source type of reflectors give a light of high intensity at the portals, free from glare and well distributed. While the light at all points within the tunnel is far short of daylight intensity, sufficient illumination is provided to enable traffic to traverse the tunnel safely without confusion or change in speed.

The change from a high intensity of light at the portals during daylight hours to a low intensity at night and vice versa is effected by an electric eye which operates a switch when the outside light intensity drops below or rises above 500 foot-candles. During daylight hours the lighting system is operated at full capacity except during sustained periods of dark, cloudy weather when the incandescent system may be cut out by manual controls. At night, since it is necessary only to provide lights for pedestrian traffic, no incandescent lights, and only one-third of the sodium vapor lights, are used.

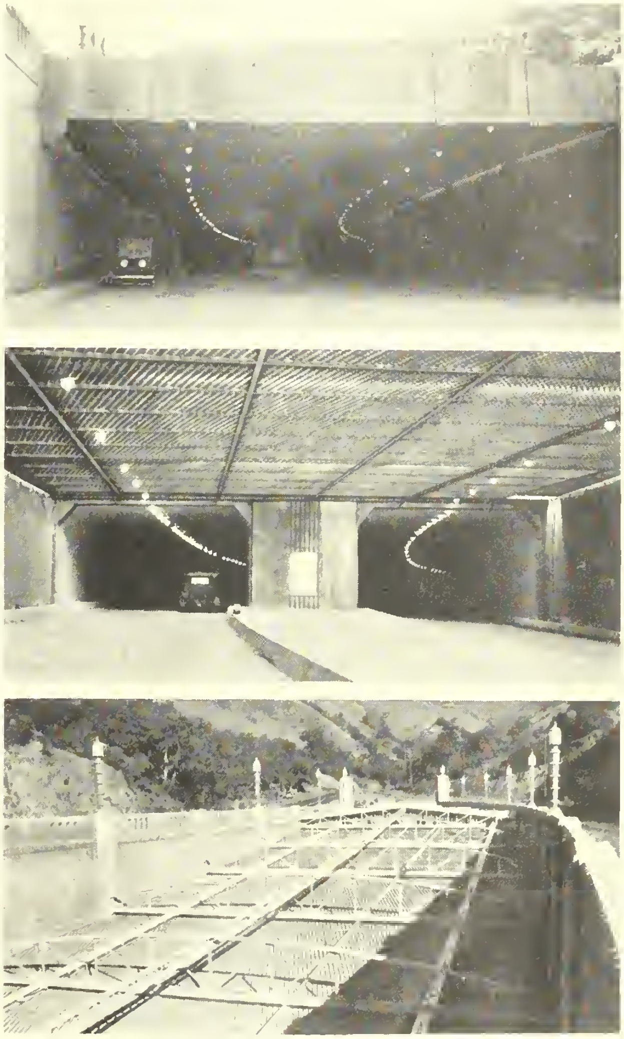

FIGURE 17.—PORTAL AND LIGHT LOUVERS OF THE BROADWAY TUNNEL, CALIFORNIA.

The photoelectric relay used is a new unit recently developed employing two phototubes instead of one, connected in parallel and mounted behind a window. This installation differs from the usual one in that the phototubes and the relay controls are located in separate boxes. The phototubes are mounted in the north wall of the vault at the west portal and the controls are located within the vault.

PLAN OF OPERATIONS IN TUNNEL CONSTRUCTION DEPENDS ON SEVERAL FACTORS

The lighting system has operated very satisfactorily, but several minor revisions would improve the design.

1. The pendant fixtures for incandescent lights near the portals do not present as good appearance as would

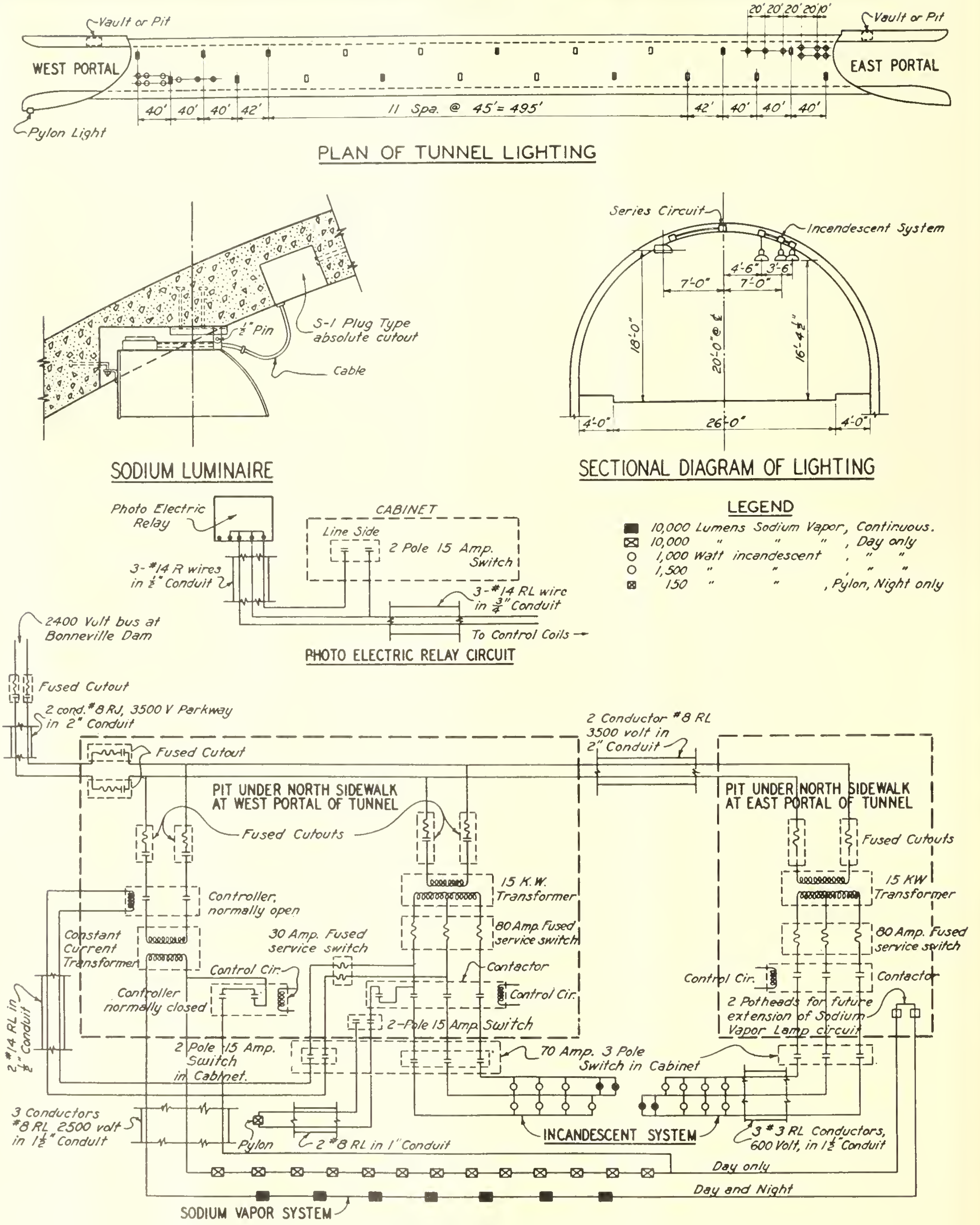
Vault or Pit
20' 20' 20' 20' 10'
Vault or Pit
WEST PORTAL
EAST PORTAL
40' 40' 40' 42'
11 Spa. @ 45' = 495'
42' 40' 40' 40'
Pylon Light
PLAN OF TUNNEL LIGHTING
1/2" Pin
S-1 Plug Type absolute cutout
Cable
SODIUM LUMINAIRE
Series Circuit
Incandescent System
4'-6" 3'-6"
7'-0" 7'-0"
18'-0"
20'-0" @ ℄
16'-4 1/2"
4'-0" 26'-0" 4'-0"
SECTIONAL DIAGRAM OF LIGHTING
LEGEND
▬ 10,000 Lumens Sodium Vapor, Continuous.
☒ 10,000 " " " , Day only
○ 1,000 Watt incandescent , " "
○ 1,500 " " , " "
⊠ 150 " " , Pylon, Night only
Photo Electric Relay
CABINET
Line Side
2 Pole 15 Amp. Switch
3-#14 R wires in 1/2" Conduit
3-#14 RL wire in 3/4" Conduit
To Control Coils →
PHOTO ELECTRIC RELAY CIRCUIT
2400 Volt bus at Bonneville Dam
Fused Cutout
2 cond. #8 RJ, 3500 V Parkway in 2" Conduit
Fused Cutout
2 Conductor #8 RL 3500 volt in 2" Conduit
PIT UNDER NORTH SIDEWALK AT WEST PORTAL OF TUNNEL
PIT UNDER NORTH SIDEWALK AT EAST PORTAL OF TUNNEL
Fused Cutouts
Fused Cutouts
15 K.W. Transformer
15 KW Transformer
Controller, normally open
Constant Current Transformer
30 Amp. Fused service switch
80 Amp. Fused service switch
80 Amp. Fused service switch
Control Cir.
Contactor
Control Cir.
Control Cir.
Contactor
Controller normally closed
2-Pole 15 Amp. Switch
2 Potheads for future extension of Sodium Vapor Lamp circuit
2 #14 RL in 1/2" Conduit
2 Pole 15 Amp. Switch in Cabinet.
70 Amp. 3 Pole Switch in Cabinet
3 Conductors #8 RL 2500 volt in 1 1/2" Conduit
Pylon
2 #8 RL in 1" Conduit
INCANDESCENT SYSTEM
3 #3 RL Conductors, 600 Volt, in 1 1/2" Conduit
Day only
Day and Night
SODIUM VAPOR SYSTEM

FIGURE 18.—DETAILS OF LIGHTING INSTALLATION OF TOOTH ROCK TUNNEL, OREGON.

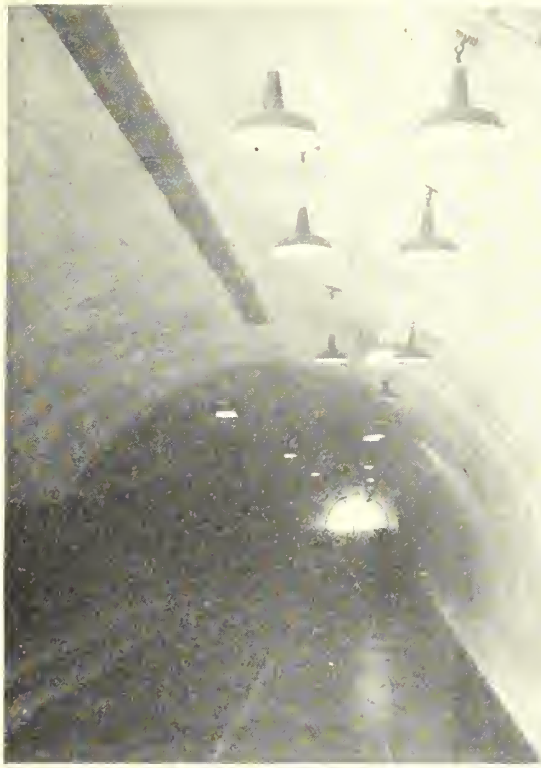

FIGURE 19.—TOOTH ROCK TUNNEL, OREGON, SHOWING LOCATION OF INCANDESCENT LIGHT UNITS.

properly concealed fixtures countersunk into the concrete lining. A more attractive design of the sodium fixtures also could be devised.

2. Probably less intensity of light at night would be equally effective for motor traffic, adequate for pedestrians and more economical. When additional sodium lights were cut out at night, the resulting lighting was very spotty and unsatisfactory. The present lighting system is also somewhat objectionable to traffic leaving the tunnel at night, since the intensity of illumination inside the tunnel is greater than that generated by headlights outside and results in a moment of deficient vision. A system of night lights of lower intensity, but spaced closer, would be more effective. A similar result could be obtained by extending the illumination outside of the tunnel to effect a transition. This transition effect will be accomplished at the Tooth Rock tunnel when the adjacent sections of road are lighted in the near future.

3. It is believed that the transition lighting during daylight would be more effective if the first units were located 20 or 25 feet inside of the portal rather than 8 feet as in the present installation. The reflected light from the outside daylight so greatly outshines the artificial light from the first units that very little benefit is derived.

The methods and sequence of tunnel construction operations depend upon the degree to which the ground is self-supporting, free, or charged with water, and on the cross section of the tunnel.

As the East Rim Road tunnel was only 22 feet wide over-all and as the sandstone formation in general was entirely self-supporting, a pilot tunnel 9 feet wide and 8 feet high was driven on center line and at grade, followed by ring drilling.

The Wawona tunnel, 28 feet wide, was in excellent, self-supporting, granite formation. The contractor tried three methods of operation: (1) A crown heading with ring drilling and bench; (2) a narrow bench and crown heading; and (3) two successive headings and one bench. The last method was used throughout the major portion of the tunnel.

Big Oak Flat tunnel No. 3 was in the same type of rock formation as the Wawona tunnel but no headings were used and the full face was taken out. In tunnels Nos. 1 and 2 a pilot heading was used for ventilation.

The plan of operations—whether to take out the entire face or one or more headings or benches—depends upon several factors.

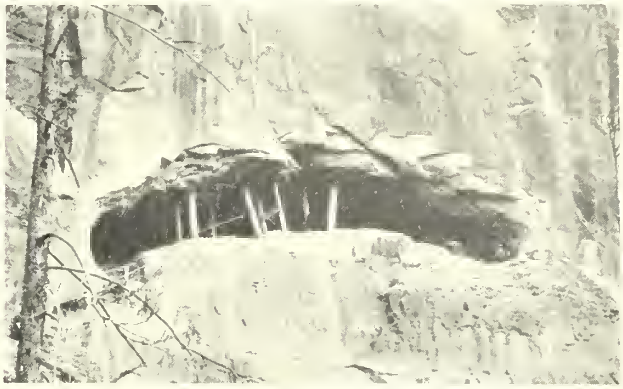

FIGURE 20.—GALLERY UNDER CONSTRUCTION ON THE EAST RIM ROAD TUNNEL, UTAH.

In the East Rim Road tunnel the boring could be done from several galleries (see fig. 20), but the major portion of the muck had to be taken out through the west portal. Therefore, to expedite completion, the pilot tunnel was well adapted to the purpose. The 22-foot over-all width made it possible to use a 9-foot by 8-foot pilot tunnel.

The pilot tunnel was worked simultaneously between the several galleries. When the pilot tunnel was completed between the west portal and the first gallery, full-bore operations were started because ring drilling, loading, and shooting could proceed at the same time as mucking. The pilot tunnel to a large extent solved the ventilation problem. The natural draft through the pilot tunnel was augmented by the exhaust from the shovel which was powered with compressed air, and no difficulty was experienced in using gasoline motor trucks as hauling equipment. In the Maricopa-Ventura tunnels and the Big Oak Flat tunnels Nos. 1 and 2, pilot tunnels were used for ventilation, but were not ring drilled. These tunnels were of short lengths and this method was used so that special ventilation machinery would not be needed.

The cross section of the pilot drift must be ample to allow easy handling of the long steel drills for subsequent ring drilling. The drill holes should extend at least a foot beyond the neat line of the section at the extreme points if the entire section is to be taken out in the two operations, that is, the pilot tunneling followed by ring drilling, loading, and blasting.

The placing of concrete lining in highway tunnels, in the past, has presented difficult problems.[5] Severely restricted working conditions and the necessity that the concrete reasonably fill all voids resulting from overbreak and areas between timber sets that must be left in place, etc., have dictated the use of extremely wet mixtures. Honeycombed areas and a rather pervious concrete have been the natural results.

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[5] The following discussion of concrete lining in highway tunnels is by G. W. Mayo, Senior Highway Bridge Engineer, United States Bureau of Public Roads.

The tunnel cross section generally permits the use of forms in the shape of a full-centered arch of comparatively short span. Structural requirements to carry the superimposed loads would permit a thin arch section but practical considerations in placing concrete generally require a somewhat thicker lining.

In recent years there has been developed a pump of the plunger type that is capable of handling dry, harsh mixtures of concrete. The use of this pump, together with vibrators both of the internal and external type, make possible the placing of tunnel lining equal in quality to concrete placed under ordinary conditions. Such equipment was used with excellent results on the following tunnels: Broadway, Yerba Buena, and Waldo, in California; Tooth Rock, in Oregon; and East Rim Road, in Utah. Tests of the 28-day strength of concrete placed in the Broadway tunnel showed that it averaged well above 3,000 pounds per square inch in compression.

PLACING PNEUMATICALLY APPLIED MORTAR REQUIRES SPECIAL CARE

Concrete pumping equipment is available in either mobile or stationary units. It has performed satisfactorily using up to the equivalent of 1,000 feet of pipe where no lift was involved. On tunnel work it ordinarily discharges directly into the forms, obviating the use of chutes or other distribution devices and consequently eliminating segregation of the aggregate from the mortar. The forms must be set in comparatively short sections and must be of substantial construction to permit the efficient use of internal vibrators and to withstand the pressures developed in pumping.

Where heavy underground pressures develop, or where the material through which the tunnel is being driven slakes rapidly in air, it is essential that the lining be kept comparatively close to the excavation heading. Experience has indicated that even though a fully lined tunnel is not contemplated, the section should be so designed as to permit the placing of lining without interference from timber sets which may need to be left in place. Costly changes in plans will thus be avoided.

Where pneumatically placed mortar lining has been placed in tunnels after the bore is complete, certain precautions are necessary to insure satisfactory results. The most important precaution is the control of air currents and temperatures during operations. In the Wawona tunnel such control was accomplished by hanging a heavy canvas curtain.

Adequate lighting is also of importance, since uniformity of application is only possible when the nozzle man is a skilled operator and all unnecessary shadows are eliminated. This lighting was much more of a problem on the Wawona than on the East Rim Road tunnel, since in the latter the surface broke smoothly while the surface of the Wawona tunnel was decidedly rough.

It is important that the rock surfaces be clean and free from dust and encrustations. The Wawona tunnel surface was washed with a stream of water under a nozzle pressure of 70 pounds per square inch. Washing was done at least half an hour before mortar was applied.

The amount of pneumatically placed mortar overran the estimated quantity in the Wawona tunnel for 3 reasons, as follows:

1. Bulking of the sand. The specifications limited the moisture content of the sand between 4 percent and 8 percent but the mix was designed on the basis of dry sand.
2. The excessive amount of rebound material obtained because of the very irregular rock surface. This rebound resulted in more sacks of cement being used per cubic yard in place than had been anticipated.
3. The actual surface area covered was considerably in excess of the area estimated, because of irregularities.

During construction of the Wawona tunnel several important features were observed in the operation of the pneumatic equipment.[6]

The cement gun was not equipped with an air gage or velocity meter, so some difficulty was experienced in maintaining a constant air pressure at the gun. The gage was located on the compressor about 40 feet from the gun. The air pressure was fairly constant at 45 pounds per square inch but frequent fluctuations were observed, caused by clogging of the material at the gun outlet or throughout the length of hose. These fluctuations caused variations in the rate of discharge of the mortar, resulting in poor hydration at the nozzle and a consequent greater loss of material by rebound. The fundamental cause of the material clogging was high surface moisture content of the sand which varied between 6 percent and 8 percent at the time observation was made. When the surface moisture content dropped below 6 percent no further trouble was experienced.

Indications are that for efficient operation the surface moisture content of the sand should be between 3 percent and 6 percent. Material containing less than 2.5 percent moisture will discharge too quickly through the gun and will not receive sufficient water for proper hydration.

It was found impractical to operate with more than 150 feet of hose. Greater length caused stoppage and resulted in considerable delay even though the air pressure was proportionately increased. When using over 100 feet of hose the contractor used a 1-inch nozzle and when using less hose a 1¼-inch diameter nozzle was used. The use of the larger nozzle resulted in a better grade of pneumatically applied mortar with considerably less rebound.

The material was batched and mixed continuously. To avoid any possible chance of hydration of the cement before it was shot through the gun, the mixing box and gun were emptied and cleaned every hour.

As has been stated, considerable material was lost because of rebound. There seems to be no particular reason for wasting this material except where linings of high strength are desired. If precautions are taken to prevent water from coming in contact with the rebound material there is no reason why it should not be used again, if collected promptly. The strength of the resulting mortar might be affected by this reuse but the saving in material cost would justify a somewhat decreased working stress. This reuse would apply to any project where pneumatically applied mortar was employed exclusively for protective purposes.

It was found highly important for the rock surfaces to be clean and free from dust before applying mortar

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[6] Further operations in the Wawona tunnel are described by E. B. Payne, Junior Highway Engineer. United States Bureau of Public Roads.

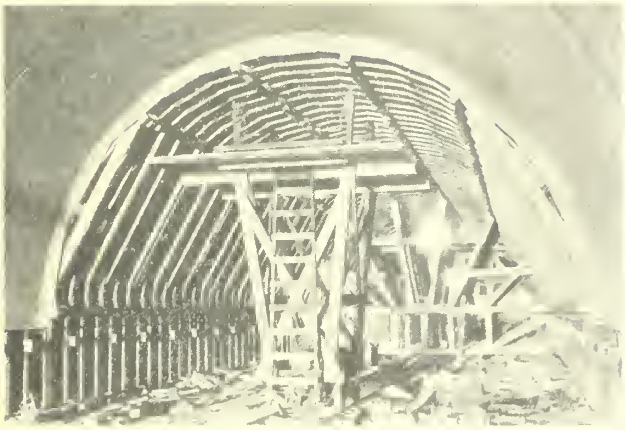

FIGURE 21.—A SECTION OF THE WAWONA TUNNEL, CALIFORNIA, SHOWING THE FORMS AND COLLAPSIBLE CENTERING USED IN PLACING CONCRETE LINING.

to them. In some instances the material sagged because of insufficient bond and at times dropped completely away from the rock. In such cases the spots were given another application of mortar.

CONSTRUCTION OF PERMANENT LINING IN HEAVY GROUND DIFFICULT

A thickness of ¾ inch is apparently the maximum that can be easily applied directly overhead when using ordinary portland cement. If a greater thickness is desired in one coat, 3 percent to 5 percent of calcium chloride should be added to the dry mixture or should be placed in the mixing water.

If the cover is generally self-supporting, lining a tunnel is a simple operation; but where heavy ground is encountered and temporary timbering is necessary, unusual care must be exercised in selecting means of supporting the lining. In the North Pacific Forest Highway tunnel in Idaho, the cover was so heavy that the timber supports were pushed into the lining section to such an extent that the original plan of concreting the timber supports in place had to be abandoned. It was necessary to place reinforced concrete rings between the sets, and later remove the sets and fill in the spaces between the rings with reinforced concrete. Note the overstressed timber posts and square sets in figure 6. Also note the reinforcing steel in place preparatory to concreting between the rings (fig. 7).

Where the cover is generally self-supporting, collapsible forms can be used, allowing easy movement as the concreting progresses. Figure 21 shows movable forms used in the Wawona tunnel. The carrier with lifting jacks in place on the raising platform is shown, also the strutting used during the placing of the concrete.[7]

The tunnel required concrete lining for 590 feet next to the upper portal. The thickness of lining was 18 inches at the springing line and 12 inches at the crown. The concrete was reinforced with ¾-inch square bars on 12-inch centers both transversely and longitudinally.

Bureau regulations against excessive drop and running of concrete in the forms led to the adoption of a rather elaborate placing system. Wooden forms, constructed in 10-foot sections and so built that they could be collapsed for moving to adjacent sections, were used. Additional centering, required while pouring, could be removed after 2 days. The forms were left in place for 5 days.

Aggregates were transported in wheelbarrows to the mixer which discharged directly into the placing gun. The discharge pipe from the gun was carried over the top of the forms, terminating in steel chutes which were carried down the sides of the forms to within 5 feet of the bottom. These chutes were in sections which were removed as the concrete built up. The entire placing installation was carried on wheels on a central track and was moved up and down the track by means of a hoist.

Thirty feet of lining were poured every 2 days, using the intervening day to move the forms and bring in materials. By moving the entire installation, the discharge boxes could be moved back and forth over the 30 feet that was being poured, thus depositing the concrete directly into final position. Tamping of the concrete was possible in all but the crown of the arch. In addition, air vibrators were used on the face of the forms.

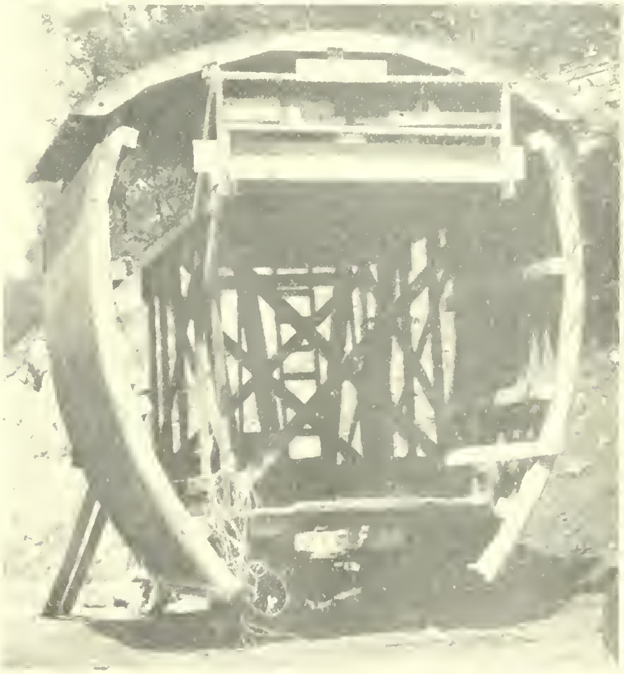

FIGURE 22.—COLLAPSIBLE STEEL FORM USED IN LINING THE BIG OAK FLAT TUNNEL.

DATA GIVEN ON CONSTRUCTION OF EAST RIM ROAD TUNNEL

At one place there was a rock fall of about 105 cubic yards from the top of the tunnel. The highest point of this pocket was 17 feet above the top of the tunnel. Through the section affected by this fall, the concrete lining was thickened to 18 inches all around the arch (see fig. 9). Three vertical pipes, extending to the top of the hole, were placed through the lining before pouring. After the concrete lining had cured a sufficient time, the entire hole was pumped full of wet sand to support the rest of the rock and form a cushion above the concrete lining.

On the Big Oak Flat tunnel, improvement was made in centering the lining by using steel shapes, and the collapsible lining was hinged in two places, as shown in figure 22.

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[7] The use of these forms and the placing of concrete are discussed by T. M. Roach, Associate Highway Engineer, United States Bureau of Public Roads.

From the East Rim Road tunnel, 5,613 feet long including the length of the six galleries, approximately 72,000 cubic yards of muck were removed.[8] Two hundred ninety-two thousand pounds of 40 percent powder were required, or an average of 4 pounds to the cubic yard.

The general operations consisted, as previously mentioned, of constructing a pilot tunnel on center line and grade (see fig. 5). Where poor cover was encountered, the location of the pilot tunnel was changed to a top heading of the same cross section along the center line.

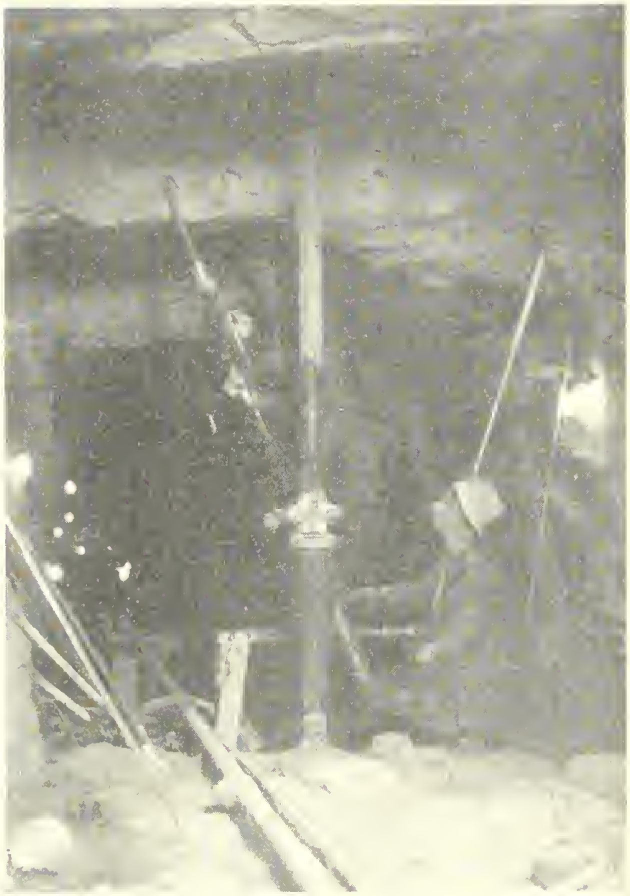

Figure 23.—Column Used in Ring Drilling, and Stoper Drill Being Operated in the Construction of the East Rim Road Tunnel.

A central compressor plant with extensive pipe system permitted operations at several galleries or pilot tunnel headings at the same time. The points of attack from the galleries and progress are shown in table 2.

The numerous crews engaged worked a total of 1,275 8-hour shifts in 273 calendar days. The average footing made each 8-hour shift was 4.4 feet, or 20.6 feet per calendar day by the combined shifts. The section of the pilot tunnel involved a quantity of 2⅔ cubic yards per foot of length.

The drilling in the side galleries and the pilot tunnel was carried on with jackhammers, column drills, or liners. All blasting was done using fuse. Muck was loaded by hand into mine cars, except where muckers could be taken into the headings.

All drilling of the material between limits of the pilot bore and the perimeter of the main bore, except where timbering was necessary, was done by ring drilling. (See fig. 23.) Stoper drills were set on vertical columns on the center line of the main tunnel, and anchored between the roof and the floor of pilot tunnel. Stopers were thus set at points 4 feet above designated grade and radial holes were drilled around the pilot tunnel for the full tunnel section above the floor. These holes were drilled to the perimeter of the theoretical bore (see fig. 5 for details) and the rings of holes were spaced at 3-foot intervals. In blasting the material in the main bore, 12 rings of holes were shot at the same time.

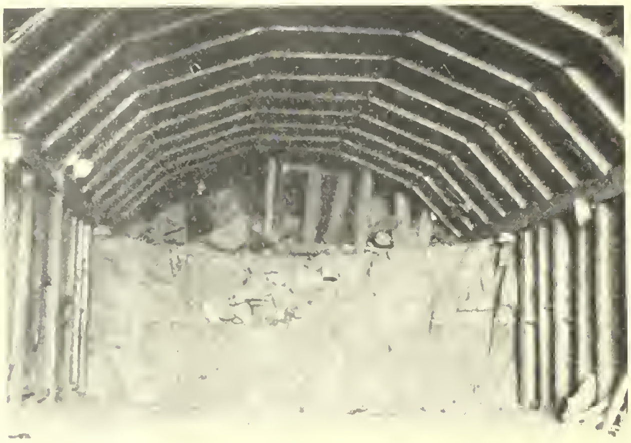

Figure 24.—Timbering Being Carried Forward as Excavation Progressed in the East Rim Road Tunnel.

Table 2.—*Progress in the construction of the East Rim Road tunnel*

Gallery station	Direction	Working time	Length	Rate per day
		Days	*Feet*	*Feet*
71+82	West	50	527	10.5
71+82	East	22	214	9.7
69+50	do	22	292	13.27
62+48	do	48	569	11.85
49+48	West	54	731	[1] 13.53
49+48	East	57	655	11.5
37+52	West	40	543	[1] 13.57
37+52	East	53	797	[1] 15.0
29+55	do	43	882	[1] 20.5

[1] Crews on bonus.

Before awarding the contract for construction of the tunnel it was thought that the ground throughout the greater part of the tunnel would be stable enough so that only a very small amount of lining would be required. When heavy ground was encountered in the pilot tunnel, however, the pilot tunnel was discontinued on grade and a top heading on springing line grade was driven and timber lining was placed between the springing line and the roof of the main tunnel. The bench below the top heading was left in place until the power shovel was ready to excavate it. This bench then was breast drilled and shot and the shovel mucked it out. As the muck was removed, posts to support the timber lining were placed under the wall plates. (See fig. 24.) The equipment used was as follows:

6 jackhammers.
12 liners and stopers.
2 150-horsepower motors.
2 stationary compressors, capacity 850 cubic feet per minute each.
1 air (tunnel type) shovel, ½ cubic yard.
5 trucks—3 to 5 cubic yards capacity.

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[8] Construction methods used in the East Rim Road tunnel in Zion National Park are described by R. A. Brown, Associate Highway Engineer, United States Bureau of Public Roads.

1 32-inch fan.
2 mucking machines.
7 mine cars.
4 hoists.
2 portable compressors.
1 automatic drill sharpener.
4 steam pumps.
Miscellaneous track, pipe, electrical equipment, and small tools.

TIMBER LINING REPLACED WITH STEEL AND CONCRETE LINING

Five years after completion of the East Rim Road tunnel it became evident that the timber lining in several sections was in distress and that it was necessary to place some permanent type of lining. In the most hazardous sections, structural steel and concrete were used as follows.[9]

The failure of the timber lining was evidenced by shifting and split arch caps and segments, deformed and crushed wall plates, and tipped posts. Sufficient timber lining was removed to make room for one 10-inch steel I-beam. (See fig. 1.) An I-beam ring was then installed, after which the sandstone outside of the beam was stulled and blocked and sufficient additional lining was removed to install another beam. Thus the operation was carried on until the entire timber section was replaced with the I-beams thoroughly wedged and the seamy, blocky, and loose sandstone above blocked and stulled in place.

After each section was completely supported with the steel I-beams, forms were placed on the inner face only and concrete was pumped between the I-beams and in contact with the rock, except that the larger voids were filled with old timber, rock, or sand to form a cushion. The concrete was placed by a pump of the plunger type.

Because of the narrow spaces of 10-inch minimum thickness in this special lining of welded I-beam sets, it was necessary to use concrete having a 5-inch slump to eliminate the possibility of leaving voids between the I-beams. The concrete was efficiently placed by skilled crews. The 10-inch steel I-beams came in three sections—two steel wall posts and an arch of steel—and were welded together to form one supporting ring. The sections were butt welded with a minimum $^3/_{16}$-inch bead completely around the joint except on the inaccessible back face.

The joints were at the springing lines and the steel ribs thus formed were placed as the sections of failing timber were removed. The steel posts were set to line and grade; the steel arch member was placed and the joints welded; and then concrete pedestals were poured at the bases of the posts. After the concrete had set sufficiently, using extra cement for early setting, the loose, blocky, and heavy ground over the arch and at the sides was blocked, wedged, and stulled to make a tight back and rigid support before filling the spaces between the I-beams with concrete. At several points it was necessary to place stulls from the tunnel floor to support the load until the I-beam could be erected and welded, and the pedestal concrete had set sufficiently.

Several old rings of 12- by 12-inch timber were found as much as 2 feet too low and were supporting heavy ground that was at many points also too low. It was necessary to cut back to obtain sufficient clearance to place the I-beam. Though the work was extremely hazardous, no one was injured. Experienced tunnel men, steel workers, and carpenters performed the work. The installation of the I-beams proved an ingenious method of repair for these sections where the timber had failed, because without steel I-beams or some temporary support, the placing of concrete lining in these sections would have been very slow and extremely dangerous. The repair work would probably have cost considerably more by other methods.

Several methods can be used in excavating tunnels. The contractor should select that method best adapted to the particular project, considering the nature of the formation, the size of tunnel cross section, and disposal of the material. The various systems of drilling used in several western tunnels are illustrated in figures 5 and 25. A detail discussion of the methods used in boring the tunnels in Yosemite National Park follows.[10]

Some of the factors which influence the choice of a method of driving a tunnel should first be considered. When driving tunnels of the size required for highways, most contractors prefer to carry a heading (this term is used here to describe a drift along the direction of the bore of such size as to permit one or more miners to excavate material within the cross section of the tunnel) some distance in advance of the excavation of the full tunnel section for the following reasons:

1. The heading discloses the character of the ground prior to opening the full tunnel section.
2. The heading offers access to points from which the crown can be reached and timbering started if required.
3. Short drill columns and drill bars can be used in the heading, thus eliminating the necessity for a large drilling setup and high working platforms.
4. Short blasting rounds are usually used in advancing a heading, and the loss is comparatively small if the blast fails to break to the full depth of the drill holes.
5. In hard rock, the blasting of the bulk of the tunnel section is more effective if there is a hole, such as a heading, toward which the material can break.

Other factors influencing the choice of methods are time allowed, equipment on hand, shape of tunnel section, etc.

CONSTRUCTION OF WAWONA TUNNEL DESCRIBED IN DETAIL

The tunnel section used in the Wawona tunnel is a semicircle, 14 to 15½ feet in radius, on a 6-foot springing line. The effective width of a top heading in such a section is reduced, as the top of the arch drops rapidly beyond 5 or 6 feet each side of the center, and interferes with the proper pointing of the drill holes. Lowering the heading from the crown of the tunnel would destroy its effectiveness as a starting point for timbering, and result in blocking of the end of the heading by the muck pile if long bench rounds had been blasted.

Three drilling systems were used by the contractor in the Wawona tunnel, and it will be noted that each system involved the use of some form of heading. (See fig. 25.)

The first method was used for the first 335 feet. Under this system, the crown heading (10 feet by 11 feet) was advanced as fast as possible. Back of the heading, but well ahead of the bench, four drilling machines, swung on cross arms from a horizontal bar, drilled holes in parallel rings 3 feet apart to break out the remaining portion of the heading to the level of the

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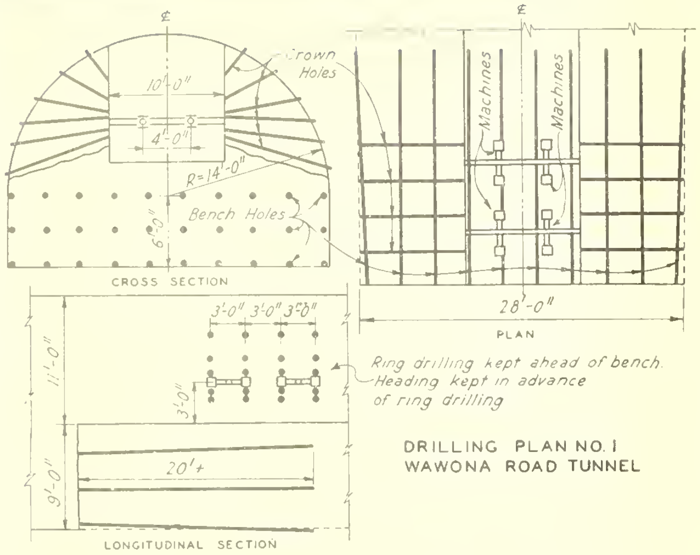
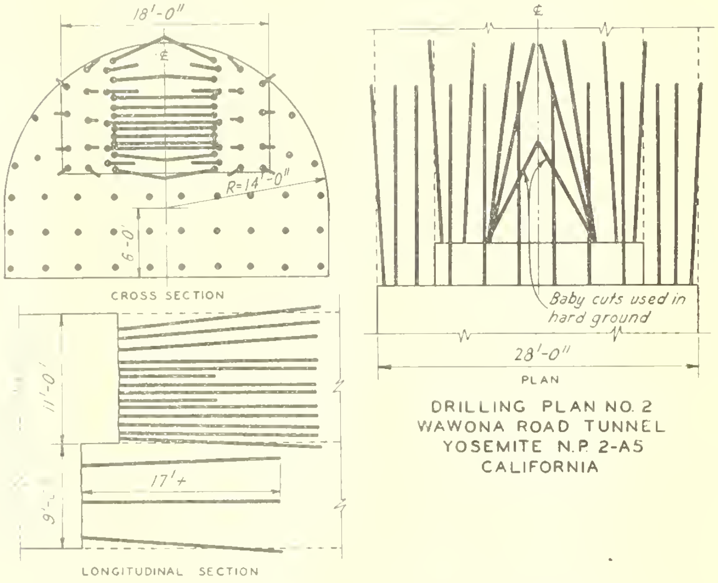
[9] This work is described by F. L. Davis, Assistant Highway Engineer, United States Bureau of Public Roads.

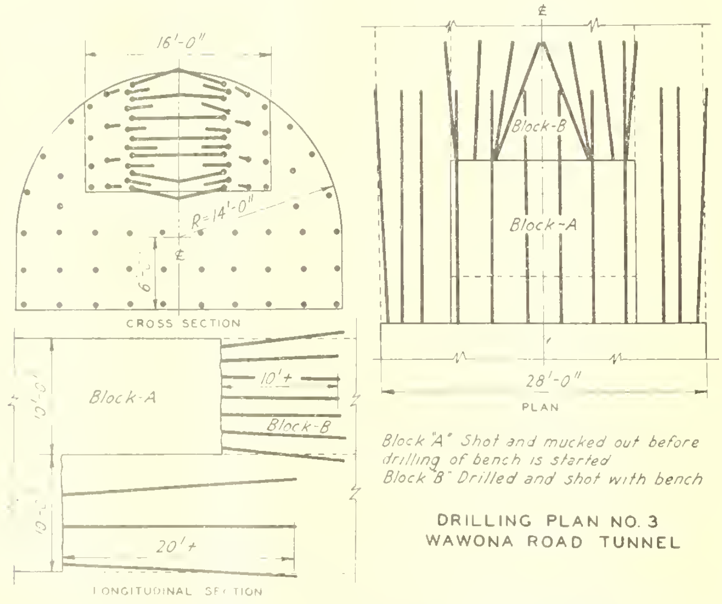
[10] This discussion is by T. M. Roach, Associate Highway Engineer, United States Bureau of Public Roads.



DRILLING PLAN NO. 1
WAWONA ROAD TUNNEL

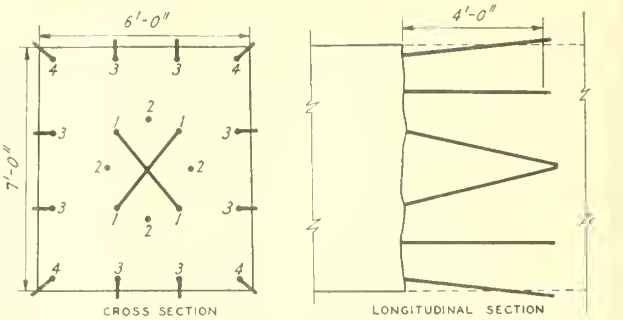


DRILLING PLAN NO. 2
WAWONA ROAD TUNNEL
YOSEMITE N.P. 2-A5
CALIFORNIA

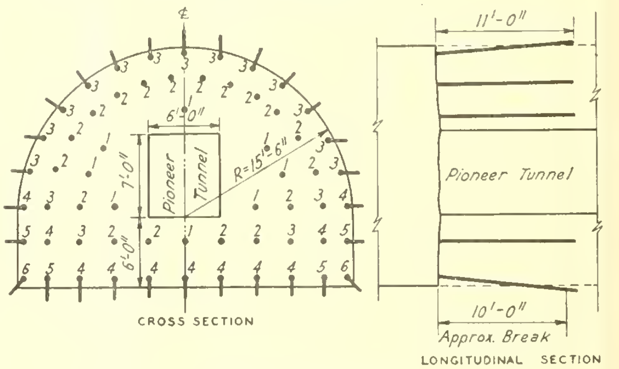


Block "A" Shot and mucked out before drilling of bench is started
Block "B" Drilled and shot with bench

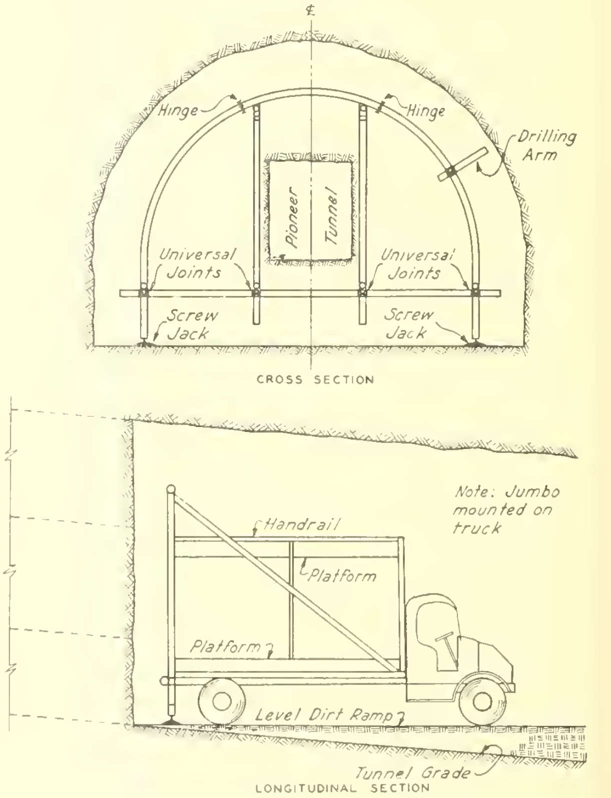
DRILLING PLAN NO. 3
WAWONA ROAD TUNNEL



DRILLING PLAN
PIONEER TUNNEL
BIG OAK FLAT ROAD



DRILLING PLAN
FOR FULL BORE
BIG OAK FLAT ROAD
YOSEMITE N.P. 3-A1 & 3-A5
CALIFORNIA



DRILLING JUMBO
BIG OAK FLAT ROAD

FIGURE 25. DRILLING PLANS FOLLOWED IN THE CONSTRUCTION OF WESTERN HIGHWAY TUNNELS; ALSO DRILLING JUMBO MOUNTED ON TRUCK.

bench. Bench holes were drilled from a cross-bar set-up, practically paralleling the tunnel center line.

In blasting, the bench hole shots were fired in conjunction with 6 or 7 rings of top holes. Delay detonators were used to assure the ring shots exploding before the bench shots, and to give proper firing order for the holes in the bench. Heading rounds were fired at the same time as the bench rounds if they could be prepared in time. The heading was mucked by hand, the muck being dumped over the bench and re-handled by the mucking rig which was used to handle the bulk of the blasted material.

It was found that the heading could not be advanced nearly so fast as the bench could be broken; that it was very difficult to keep air pipes in place past the mucking rig to feed the machines drilling in the heading; and that there was, in the heading, constant interference with the machines drilling the ring holes ahead of the bench. The daily average advance of the entire section was only 5.6 feet, and this system was abandoned in favor of the second method.

The second drilling plan utilized a heading 18 feet wide, kept only a short distance ahead of the full tunnel face. A set-up for machines drilling from short arms swung from a column was used in the heading. The drill holes varied from 13 feet to 19 feet in depth, but usually gave about a 17-foot break.

In erecting the drilling set-up for the bench, two horizontal bars were placed across the tunnel, the first 1 foot above grade, and the second at the springing line. These bars were tightly jacked against the rock sidewalls. Clamped to these bars was a curved bar, 18 inches from the sidewall and parallel to the curve of the tunnel section, extending up into the section of tunnel beside the heading. This bar was used as a guide in drilling the area above the bench and was shifted from one side of the tunnel to the other as drilling progressed. Machine clamps could be placed on any part of the curved or horizontal bars. The drill holes were spaced as shown in figure 25 and were drilled approximately parallel to the tunnel center line to a depth of 13 to 19 feet. Heading and bench were blasted together, delay detonators being used to give the proper sequence of explosions. Charges in all holes in the heading exploded before charges in the holes in the bench.

Under the second method the maximum daily advance was 17 feet, while the average was only 13 feet. It was very difficult to remove the muck from the heading after shooting using the long holes, as they sometimes produced "bootlegging", or failure to break to the full depth of the holes. This loss sometimes ran as high as 4 or 5 feet and necessitated reshooting with consequent loss of time. Widening the heading to permit longer rounds would merely have lowered the heading sidewall height to a point where it would have been impossible to place the holes properly, and lowering the heading from the crown would have resulted in partial blocking of the heading by the muck pile and failure of the muck from the heading to blast clear of the bench.

The contractor was equipped and staffed for working 24 hours per day, doing most of the drilling and blasting in one shift and the mucking in the two remaining shifts. The mucking operations had been developed to the point where the contractor was convinced that a full 20-foot round could be cleaned out during the two mucking shifts, and he accordingly changed to the third method, which he believed would produce the desired footage.

Under this system of drilling, the contractor endeavored to shoot and remove two 10-foot rounds in the heading and one 20-foot round in the bench each 24 hours. One of the heading rounds was drilled and fired in conjunction with the bench round, the second heading round being drilled and fired during the first mucking shift. The arrangement of drill holes and firing order were essentially the same as was used in the second method.

Using the third system it was necessary to blast twice each day, each blast resulting in a loss of at least 1 hour while the smoke and gases were being cleared out. Further delay was caused by extra moving of the shovel for cleaning up the muck after each blast. It was found impossible to complete the working cycle in 24 hours, and all shifts were accordingly worked on call, their hours changing each day, with consequent disorganization.

The third drilling system was used in driving the last 3,157 feet of the tunnel. The daily average advance was 13.6 feet, although the advance per blast was 20 feet or more, as had been estimated.

The costs using the second and third drilling methods were practically the same. It will be noted that the average advance was only 0.6 foot per day more under the third plan. It is believed the costs would have been lowered slightly if the contractor had not changed to the third system but had continued the second system and had concentrated on lowering the costs by increased efficiency.

All mucking was done with a ¾-swing power shovel, mounted on crawler-type tracks and operated with compressed air. This rig was slow and cumbersome, and production was low. It was usually 3 hours after the mucking shift started before the first train of material came out of the tunnel. Operating costs were high, as the shovel would not operate on the output of one compressor of 1,250 cubic-feet-per-minute capacity, and it was necessary to run two compressors requiring 400 horsepower. The same size shovel of the conventional type will operate on one 75 to 100 horsepower motor.

Material was hauled from the tunnel in side-dump cars, operated on tracks by combination battery and trolley locomotives. In addition to the high cost of installation and maintenance of this haulage system, the equipment did not operate well on the 5-percent grade of the tunnel. The locomotives could barely push 3 empty cars up the grade, and a great deal of trouble was encountered in controlling the loaded train while coasting down grade.

It appeared that operations involving the blasting and mucking of the full face, with approximately the same daily advance or possibly less, would have been more economical.

DRIVING OF BIG OAK FLAT TUNNELS DISCUSSED

The following factors were considered when deciding upon the method to be used in driving the two short tunnels on the lower portion of the Big Oak Flat road.

1. The two tunnels totaled only 540 feet in length, so any expensive equipment set-up would have materially affected the cost.

2. The available compressor could supply air to operate only about 7 small drilling machines or four large drilling machines. Two machines that could be made over into small drifters were on hand.

3. No bad ground was anticipated, and no top heading was believed necessary. It was believed, however, that a pioneer bore should be used to provide for ventilation while enlarging to full size and to give a better break to the material while driving the full tunnel section.

4. The power shovel on hand was considered capable of mucking out only about 10 feet per day, so long rounds were out of the question.

5. All drill steel on the project was of the 1-inch hexagonal type, threaded for detachable bits. This steel could be used with small drifters, but was unsuitable for use with large machines. Use of this steel and the bits made purchasing of drill sharpeners and furnaces unnecessary.

6. The time involved in driving a pioneer bore through the tunnels would not delay completion of the project, as the pioneer bore could be driven through the first tunnel before the shovel was available to start the enlarging work. The pioneer bore through the second tunnel could be driven while the shovel was working in the section between the tunnels.

It was accordingly decided that a pioneer bore, 7 feet by 6 feet should be driven through both tunnels and that the enlarging work would be done at the approximate rate of 10 feet per day. The drilling was to be done from a frame, or "jumbo", all holes being drilled approximately parallel to the center line of the tunnel. The pioneer bore was located with its lower edge on the springing line of the tunnel section. The pioneer bores were drilled as shown in figure 25; the resulting advance was between 3.5 feet and 4 feet per blast. Two shifts were used on this work each day, and the daily advance varied between 7 and 8 feet. All mucking in the pioneer bore was done by hand, and the material was transported in a small car, hand operated on tracks. All blasting was done with electric detonators and the firing order was controlled with delays, as illustrated by the numbers on figure 25.

The drilling jumbo was mounted on a flat-rack truck, and was moved in and out of the tunnel for each drilling shift. This jumbo is illustrated in figure 25 and is made of extra heavy 4½ inch pipe, curved to follow the neat lines of the tunnel arc but having a radius 3½ feet less than that of the tunnel. The uprights and braces are fastened to the truck, while the curved side arms are hinged near the top to permit swinging them out of the way while moving the jumbo. A solid bar, 25 feet long was placed across the tunnel and solidly clamped to the curved arms, 3.5 feet above the tunnel grade, while drilling was in progress. Jack screws on the bottom of the curved arms served to lift some of the weight from the truck and to brace the entire frame solidly. Drill arms could be swung out from any part of the frame, or the machines could be mounted on any portion of the frames or braces.

Drill holes were required on approximately 3-foot centers, and were drilled by using drilling arms 4 feet long clamped to the frame with universal clamps. The truck was placed on center line and the frame jacked to grade. Six drilling machines were mounted on the frame at one time, and drilling was started at the top of the frame. The machines were moved down the frame as drilling progressed. The pointing of each drill hole was checked by the shift boss. Approximately 60 holes were required to break out each section. These holes were drilled in two shifts, the first shift setting up and starting the drilling, while the second shift completed the round and loaded and blasted the holes. The mucking was done in one shift, all material being transported in trucks.

This system was very successful in the two short tunnels, and is believed to be definitely superior to the system used in the Wawona tunnel. Powder consumption was very low, the pioneer bore requiring 14 pounds of powder per cubic yard while the enlargement required only 2.4 pounds per cubic yard. The average was 3.3 pounds per cubic yard as against about 7 pounds per cubic yard in the Wawona tunnel and an estimated 4.6 pounds per cubic yard in the 2,167-foot tunnel on the Big Oak Flat road.

Of even more importance than the cost was the effect this system had on the final shape of the completed tunnel. The material encountered was badly seamed, and considerable overbreak was unavoidable. Under this method the heavy blasting required close to the crown when a top heading is used was avoided, and only light shooting to break out the small amount of material from above the top of the pioneer bore was required. None of the rock above the crown was shattered, all overbreak being the result of material breaking away from well defined seams above the neat lines.

In driving the 2,167-foot tunnel on the Big Oak Flat road, it was decided that the drilling jumbo developed for the two short tunnels would be used, but that it would be modified to provide for platforms swinging from the main jumbo for work on the sides. This eliminated the second truck with platforms which had been used on the short tunnels.

LOADED TRUCKS PULLED OUT OF TUNNEL BY ELECTRIC HOIST

Because of restrictions on the location at which the spoil could be dumped, the Big Oak Flat tunnel was driven down grade. On down grades electric haulage, besides involving a large capital investment and installation cost, is generally not efficient and involves transfer of the material to trucks for the further haul outside of the tunnel. It was therefore decided that trucks would be used in this tunnel, avoiding the necessity of using the truck engine inside the tunnel by coasting the trucks in and pulling them out. The use of trucks, which were backed into the tunnel for loading, necessitated the use of a full revolving shovel for mucking.

The use of a pioneer bore was not practical because of the time required to put such a bore through and because electric haulage would be required. The drilling system illustrated in figure 5 was therefore adopted. It is a full face system using about 100 holes, with the cut holes, which are loaded the heaviest, kept well away from the crown of the tunnel.

A compressor set-up involving approximately 1,250 cubic feet of air per minute was required to handle the drilling and shop work. Six drills are used, each machine being of the automatic-feed, drifter type.

The drilling and blasting were done in two shifts, each shift using the six drills. Holes were drilled as shown in figure 5, the pointing of the hole and the proper distribution of electric delay detonators being checked by the shift bosses. All blasting was done by electricity, the firing switch being outside the tunnel. It was usually possible to blast about 2 hours before the mucking shift was started, thus giving ample time for ventilation of the tunnel.

Ventilation was provided by a blower with a 24-inch ventilation pipe, supplemented by a portable blower and a 12-inch flexible air pipe near the working face.

Mucking was done in one shift by a full revolving 1¼-cubic yard shovel powered with an electric motor. Power was supplied to this shovel through a special three-conductor drag cable, so constructed as to resist the abrasive and cutting action of the rock bottom.

Hauling was done with 5-cubic yard dump trucks. A double drum electric hoist was mounted outside of the tunnel portal, the cable between the two drums being endless and going over rollers on the tunnel floor and around a tail block which was kept about 40 feet from the working face. Hoisting could thus be done with either drum, and hoisting of a truck could be started as soon as the preceding truck was outside the tunnel. Each truck was equipped with a 60-foot choker cable. Trucks were coasted into the tunnel backwards, and were connected to the hoisting line, when loaded, by means of a special choker hook. After clearing the tunnel portal, the trucks were driven under their own power to the dump, which was about 1 mile from the portal.

Under this system approximately 10 feet to 12 feet were advanced per cycle, involving the full 24 hours. It is believed that the same system could be used to give a daily advance of from 18 feet to 20 feet if sufficient transmission line, transformers, compressors, and drills were used so that larger rounds could be drilled and blasted in 8 hours and the mucking carried on in two shifts, but this system would involve more than doubling the amount of equipment.

The only apparent disadvantage of this system is that if bad ground is encountered the full face is open, and timbering is very difficult. Nothing but full timber sets can be placed, and these cannot be blocked in the center of the tunnel or it is impossible to take the large equipment through. On a short stretch of heavy ground encountered in the Big Oak Flat tunnel, in order to avoid blocking the roadway to movement of equipment, the timbering was reinforced with concrete between the sets for the full depth. This concrete was built up with mortar by the pneumatic gun placement method. This heavy ground also necessitated the use of a heading. Mucking was done by hand for a short portion of the section.

The ventilation system of the Wawona tunnel is of particular interest and will be discussed in some detail.[11]

Carbon-monoxide gas is a serious menace in the operation of automobiles in tunnels. The pure gas is odorless, tasteless, and invisible. Its very toxic effect does not, however, produce a warning symptom. An ingenious ventilation system was installed in the Wawona tunnel in Yosemite Park to keep the tunnel free from too great a concentration of this dangerous gas.

Meteorological data indicated that natural draft could not be relied upon entirely as it was extremely variable. Calculations showed that carbon-monoxide gas could accumulate to dangerous concentrations during periods of heavy traffic. It was decided, therefore, to install an automatically controlled ventilation system. Natural draft was used as much as possible and was supplemented by the ventilating fans when needed to keep the carbon-monoxide concentration within safe limits.

Three horizontal adits or smaller tunnels at right angles to the main tunnel were driven to the face of a cliff to assist the natural draft. Two of these adits are 7 feet by 6 feet and 300 feet long. They are located at the quarter points or about 1,000 feet from their respective portals. The third or main adit is of the same cross section area as the tunnel and is 500 feet in length. It is centrally located, and in it are installed three 9-foot fans with a combined air delivery of 300,000 cubic feet per minute. (See fig. 26.)

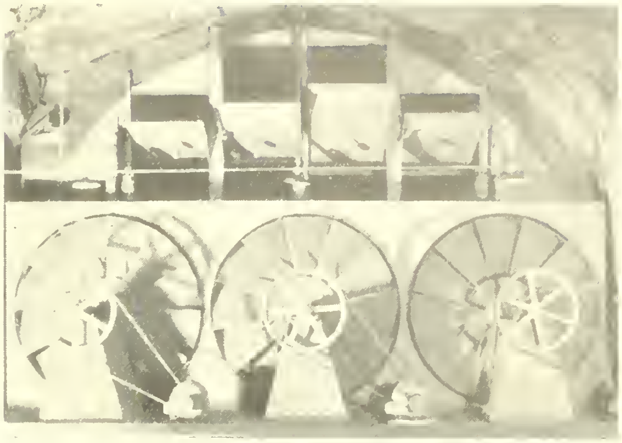


FIGURE 26.—EXHAUST AIR FANS USED IN VENTILATING THE WAWONA TUNNEL.

CHEMICAL ANALYZERS DETERMINED CARBON MONOXIDE CONTENT OF TUNNEL AIR

Each fan has 12 blades, and is of the auto vane, ball bearing, pedestal-mount type. The capacity of each fan at full speed (400 revolutions per minute) is 100,000 cubic feet per minute. At half speed, or 200 revolutions per minute, the capacity is 50,000 cubic feet per minute. The fans are connected by means of a belt to two-speed induction motors of 25 and 12.5 horsepower when operating at 1,800 and 900 revolutions per minute, respectively.

The concentration of carbon monoxide in the tunnel is determined by two chemical analyzers. Samples for each analyzer are taken through a 1-inch intake pipe, one pipe taking a sample of air in the eastern half of the tunnel and the other a sample of air in the western half. Each analyzer is equipped with a rotary air pump and draws continuous samples of air through its intake pipe. The air sample is thoroughly cleansed of all dust impurities and accurately regulated to predetermined volume before entering a hopcalite cell for the determination of its carbon-monoxide content. The concentration of carbon monoxide present in the air sample is graphically recorded by the analyzer on a moving paper chart.

In the hopcalite cell any carbon monoxide in the sample air starts action of a thermocouple which steps up a current. Then a bridge galvanometer actuates a carriage with motive power from a synchronous motor. The carriage operates 6 contact disks which, in turn, control the fans. This carriage also carries a pen which registers on a roll chart. The concentrations of carbon monoxide and the corresponding fan operation are given in table 3.

A selector switch enables the fans to be controlled either automatically or manually. When the selector switch is set to operate automatically, the automatic recorders have full control of the fans. When the switch is set for manual operation there is partial automatic control and when it is in the off position there is full manual control.

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[11] This discussion is by Walter Champion, Senior Engineering Aide, United States Bureau of Public Roads.

TABLE 3.—*Fan operation for various concentrations of carbon monoxide in the Wawona tunnel*

Parts of carbon monoxide per 10,000 parts of air	Fan operation		
	Number operating	Speed	Capacity
		R. p. m.	*Cu. ft. per min.*
0.49			
0.5	1	200	50,000
1.0	2	200	100,000
1.5	3	200	150,000
2.0	1	400	100,000
	2	200	100,000
2.5	1	200	50,000
	2	400	200,000
3.0	3	400	300,000

A more detailed description of the analyzers and the switch controls follows:

The whole sampling operation is accomplished with the equipment shown in figure 27 by passing the sample of air through a charcoal trap and a sulphuric acid bath where dust, dirt, and moisture are removed. The sample next passes through a filter tower of alternate layers of charcoal, cotton, and glass wool, where acid spray and other impurities are removed. The sample continues on through a soda, lime, and charcoal cannister that neutralizes or filters out any remaining acid fumes, gases, and impurities. The sample now passes into the flow meter where it is regulated to a continuous volume of 48 liters per minute. The regulation is automatic, since any increase in resistance to the left of the capillary orifice is transmitted to the right of the capillary tube, through the water head reservoir, water cylinder, and bubbling tube. Raising or lowering the bubbling tube changes its hydrostatic head in the water cylinder, which proportionately changes the air flow by bubbling out surplus sample air to the atmosphere. The sample next passes through a calcium chloride drying tube that removes any moisture picked up in the flow meter. (See fig. 27.)

The prepared sample then passes through a tubular heating coil immersed in the steam bath where it is warmed before entering the hopcalite cell for analysis. The hopcalite cell is also immersed in the bath, permitting the thermocouple to generate a continuous normal current. The bath is maintained at a constant uniform temperature of 208 degrees F.[12] by an electric heating element, assisted by an air-cooled condenser which removes resulting steam. The thermocouple contains 36 joints, half of which are imbedded in active hopcalite and form the hot pole, while the other half are imbedded in inert pumice and form the cold pole. The sample of air, in passing through the cell, comes in contact with the hopcalite, and any carbon monoxide present in the sample sets up a catalytic oxidation, the intensity of which is proportional to the concentration. The resultant heat reacts on the hot joints of the thermocouple, causing the generation of an electric current, the intensity of which is proportional to the degree of heat developed by the catalytic oxidation. The current thus generated in the hot pole is conducted through a bridge galvanometer, where it is measured in millivolts and returned to the cold pole, completing the circuit.

The thermocouples of the analyzers are connected to the galvanometers of their respective recorders through a bridge or split potentiometer circuit as shown in figure 27. The circuit is electrically balanced through the medium of rheostat R1 and potentiometers P1 P2, until that portion of the dry-cell current equals the normal current generated in the thermocouples by the heat action of the steam bath. When the circuit is thus in electrical balance, there will be no flow of current through the galvanometer; however, when catalytic oxidation occurs and generates a current of higher value than the normal current, the electrical balance will be upset and cause the galvanometer proportionately to deflect from zero. The current of the electrical balance is established at the time of calibration of the analyzers. The electrical balance is checked or compared with this value every 30 minutes by the automatic closing of cam switch S1, which connects the standard or mercury cell into the circuit comparing the dry-cell voltage across resistances R2–R3–R4 R5, and compensating any differences by adjusting R1 and P1. The galvanometer draws about 10 millivolt-amperes for full scale deflection or registration of 10 parts of carbon monoxide. The dry-cell voltage of 1.5 volts is reduced to 1 volt through the rheostat R1. When the cell voltage drops to 1 volt the cell should be replaced. The standard, or mercury cell, is in the circuit only half a minute in every half-hour and should last indefinitely.

When concentrations are below 0.5 part, the pen carriage contacts, H1 to H6, are open and their associated C.L. contacts are closed, and relays R1 to R12 are de-energized as shown in figure 27. As concentrations increase and the pen carriage moves up the scale, the contact disks 1 to 6, progressively, open their C.L. contacts and close the associated H contact. Assume recorder A pen carriage to be moving up the scale closing contacts H1–H2–H3. As the contacts close, their respective operating coils Nos. 1–2–3 of low-speed relays R1–R2–R3 are energized and pull in, energizing in turn the operating coils L1–L2–L3 of line contactors R10–R11–R12, which "Y" connects the fan motors to 8 poles and operates them on low speed or 900 revolutions per minute. As the pen carriage continues up the scale and closes contacts H4–H5–H6, their associated operating coils Nos. 4–5–6 of high speed relays R7–R8–R9 are energized and pull in, dropping out the low speed relays and line contactors, and energizing the operating coils H1–H2–H3 of line contactors R10–R11–R12, which delta connects the fan motors to 4 poles and operates them on high speed or 1,800 revolutions per minute.

When the concentrations decrease and the pen carriage returns down scale, the "H" contact will open, first followed by the closing of its respective C.L. contacts. As the contacts C.L.6–C.L.5–C.L.4 close, the holding current of relays R9–R8–R7 through their contacts A1 will be by-passed around the operating coil through the C.L. contacts to the coil resistor and LX2, causing them to drop out and open the line contactors' high-speed operating coils, removing the motors from the line. As the high speed relays R9–R8–R7 open, the time delay relays R4–R5–R6 start and run through a pre-set time of 5 minutes, allowing the fans to coast down from 400 to 200 revolutions per minute, before permitting the low speed relays R1–R2–R3 to pull in and apply power to the motors for operation on low speed. If the pen carriage, in coming down the scale, closes the contacts C.L.3–C.L.2–C.L.1, before their respective time delay relays run out the time, the fans will coast on down to rest without going on low speed. The time delay relays are adjustable from 0 to 33 minutes and are used to prevent reactive current and strain on the fan blades. Recorder B is an exact duplicate of recorder A and controls the relays R1–R2–R3–R7–R8–R9 through the duplicate operating coils 1.1 to 6.1, which in turn control the same line contractors and time delay relays as did recorder A. The recorder registering the highest concentration takes precedence in governing the fans to that concentration.

At 48-hour intervals, the 8 pounds of sulphuric acid in the air-scrubbing train in the analyzer is renewed, and about 1 pint of water added. The chart rolls of the graphic recorders are replaced once a month. The drying tubes are repacked every 4 months with 2 pounds of anhydrous lump calcium chloride. The canister is refilled twice a year with a mixture of soda lime and charcoal. The charcoal trap and tower filter are refilled once a year with activated lump charcoal. The hopcalite cell is repacked twice a year with approximately 1.7 ounces of 14-mesh hopcalite and 0.48 ounce of 14-mesh pumice, after which it must be recalibrated. Pure carbon monoxide gas for calibrating the analyzers and recorders is manufactured in the control room by slowly dripping formic acid on hot sulphuric acid. Electric lamps last for about 1,800 hours of burning and are replaced as they burn out; reflectors are washed twice a year.

VENTILATING AND LIGHTING COST DATA GIVEN

The motor grouping switch is manually controlled and has three positions. When set on the first position fan No. 1 comes on first followed by Nos. 2 and 3. When set on the second position fan No. 2 comes on first followed by Nos. 3 and 1. When set on the third position fan No. 3 comes on first followed by Nos. 1 and 2. The switch is shifted to a new position once a week; thus distributing the wear on the fans and motors, otherwise, one or two fans would receive most of the wear.

Each portal is equipped with a semaphore traffic warning signal which automatically drops to the stop

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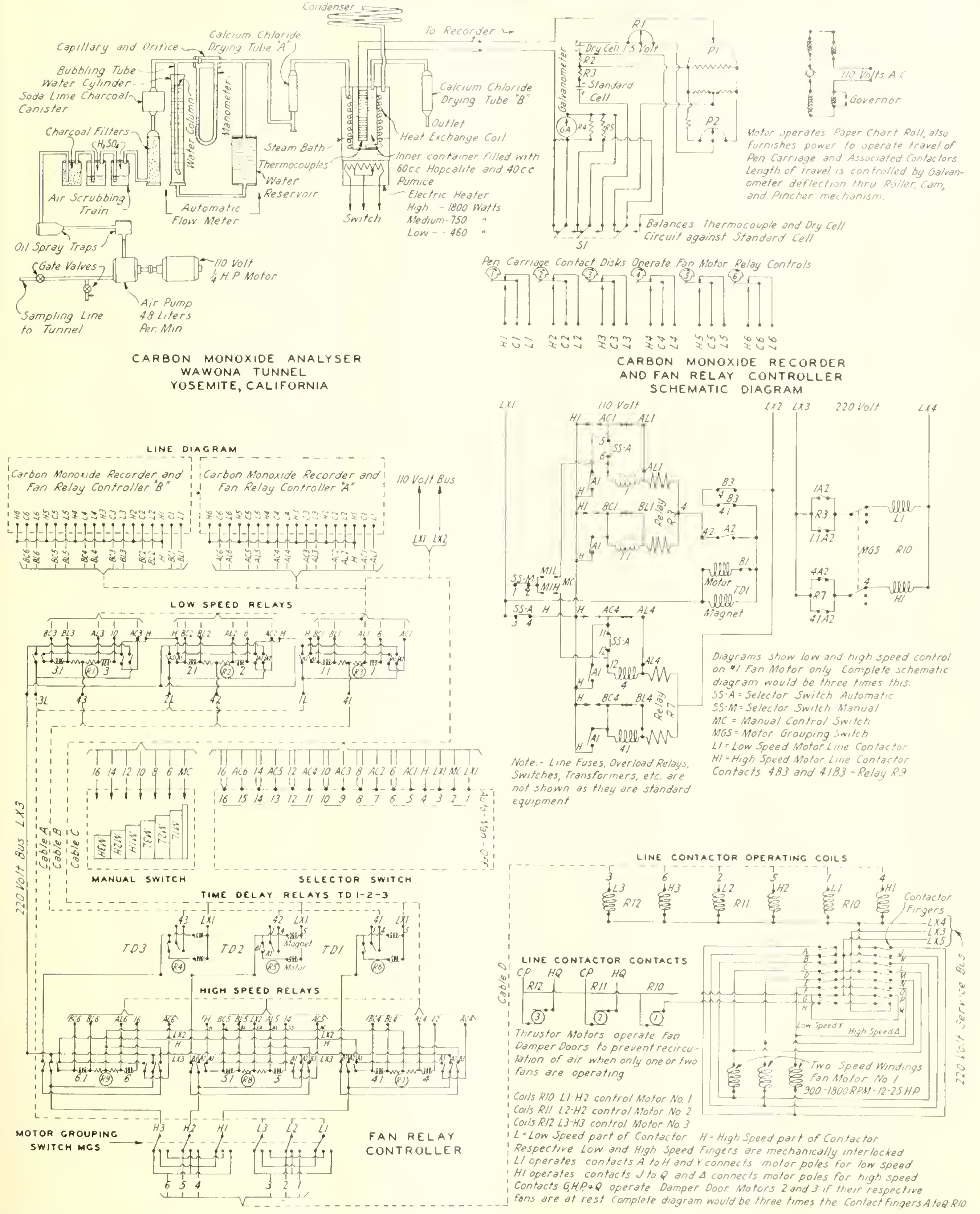
[12] This is above the boiling point of water at this elevation.



CARBON MONOXIDE ANALYSER
WAWONA TUNNEL
YOSEMITE, CALIFORNIA

Motor operates Paper Chart Roll, also furnishes power to operate travel of Pen Carriage and Associated Contactors. Length of travel is controlled by Galvanometer deflection thru Roller, Cam, and Pincher mechanism.

Balances Thermocouple and Dry Cell Circuit against Standard Cell

CARBON MONOXIDE RECORDER
AND FAN RELAY CONTROLLER
SCHEMATIC DIAGRAM

Diagrams show low and high speed control on #1 Fan Motor only. Complete schematic diagram would be three times this.
SS-A = Selector Switch Automatic
SS-M = Selector Switch Manual
MC = Manual Control Switch
MGS = Motor Grouping Switch
L1 = Low Speed Motor Line Contactor
H1 = High Speed Motor Line Contactor
Contacts 4B3 and 41B3 = Relay R9

Note.- Line Fuses, Overload Relays, Switches, Transformers, etc. are not shown as they are standard equipment

Thrustor Motors operate Fan Damper Doors to prevent recirculation of air when only one or two fans are operating

Coils R10 L1-H1 control Motor No. 1
Coils R11 L2-H2 control Motor No. 2
Coils R12 L3-H3 control Motor No. 3
L = Low Speed part of Contactor H = High Speed part of Contactor
Respective Low and High Speed Fingers are mechanically interlocked
L1 operates contacts A to H and Y connects motor poles for low speed
H1 operates contacts J to Q and Δ connects motor poles for high speed
Contacts G,H,P,+Q operate Damper Door Motors 2 and 3 if their respective fans are at rest. Complete diagram would be three times the Contact Fingers A to Q R10

FAN RELAY CONTROLLER

FIGURE 27.—SCHEMATIC DIAGRAMS OF CARBON MONOXIDE ANALYZERS, CARBON MONOXIDE RECORDER, AND FAN RELAY CONTROLLER INSTALLED IN THE WAWONA TUNNEL.

position if carbon monoxide concentrations become excessive. Telephones are installed at each portal and in the center adit. They all are connected to the National Park Service switchboard in Yosemite National Park and provide a means of communication when necessary.

Electrical energy for light and power is supplied by a hydroelectric plant at 2,300 volts, 60 cycles, 3-phase. It is transmitted to the tunnel through an overhead transmission line carried on steel poles. The transmission line is of considerable interest. It is approximately 8,000 feet long and has numerous spans of 400 to 600 feet between poles. One span is 1,200 feet long, with a difference in elevation of supports of 850 feet. A control room located in the central, or fan adit, houses the analyzers, recorders, switchboard, transformers, etc. The total load is 98.6 kilowatts, distributed as follows:

	Kilowatts
Tunnel lighting	32. 1
Ventilation	57. 5
Analyzers, recorders	6. 5
Signals, relays, controls	2. 5
Total	98. 6

Inspection of all the apparatus is made every 48 hours.

The necessity of artificial ventilation to augment natural draft is borne out in table 4 which shows carbon monoxide gas concentrations and traffic data for a 12-hour period on May 30, 1937.

TABLE 4.—*Carbon monoxide gas concentrations and traffic data for a 12-hour period on May 30, 1937 (Wawona tunnel)*[1]

Time	Carbon monoxide in parts per 10,000 parts of air	Traffic
6 to 7 a. m	0. 3	125
7 to 8 a. m	. 4	200
8 to 9 a. m	. 5	175
9 to 10 a. m	3. 3	250
10 to 11 a. m	5. 6	350
11 to 12 m	4. 4	500
12 to 1 p. m	3. 5	558
1 to 2 p. m	6. 1	615
2 to 3 p. m	3. 6	820
3 to 4 p. m	3. 9	737
4 to 5 p. m	2. 0	610
5 to 6 p. m	2. 6	400
Average or total	3. 0	5, 340

[1] Total power consumption for ventilation, lights, etc. for 24-hour period was 1,150 kilowatt-hours.

The total construction cost of the tunnel amounted to $563,729.31. Of this amount, $39,693.33, or 7 percent, was expended for ventilation and lighting equipment distributed as follows:

Carbon monoxide analyzers, recorders, etc	$10, 061. 49
Ventilation fans, motors, transformers, etc	8, 394. 95
Lighting units, transformers, conduits, etc	6, 440. 89
Automatic switchboard, relays, controls, etc	2, 184. 00
Telephones, cable transmission line	1, 913. 00
Warning signals, devices, cable, etc	1, 555. 00
2,300 volt transmission line, power panel, etc	9, 144. 00
Total	39, 693. 33

The estimated traffic for the last 11 months of 1936 was 167,830 vehicles and the operating cost amounted to $4,624.45, distributed as shown in table 5.

The estimated traffic for the first 6 months of 1937 was 102,993 vehicles and the operating cost amounted to $1,964.45, distributed as shown in table 6.

TABLE 5.—*Power consumption and operation costs for lighting and ventilating Wawona tunnel, last 11 months of 1936*

Month	Power consumption for lighting and ventilation	Cost [1]
	Kilowatts	
February	13, 080	$196. 20
March	10, 700	160. 50
April	22, 360	335. 40
May	26, 810	402. 15
June	25, 270	379. 05
July	26, 240	393. 60
August	26, 220	393. 30
September	23, 020	345. 30
October	11, 870	178. 05
November	3, 680	55. 20
December	4, 580	68. 70
Total	193, 830	[2] 2, 907. 45

[1] At $0.015 per kilowatt-hour.

[2] The cost of inspection, maintenance, and repairs (labor) was $1,609; the cost of material and supplies was $108; making a total cost of $4,624.45.

TABLE 6.—*Power consumption and operation costs for lighting and ventilating Wawona tunnel, first 6 months of 1937*

Month	Power consumption for lighting and ventilation	Cost [1]
	Kilowatts	
January	14, 270	$214. 05
February	10, 460	156. 90
March	11, 420	171. 30
April	12, 290	184. 35
May	21, 300	319. 50
June	23, 890	358. 35
Total	93, 630	[2] 1, 404. 45

[1] At $0.015 per kilowatt-hour.

[2] The cost of inspection, maintenance, and repairs (labor) was $500; the cost of material and supplies was $60; making a total cost of $1,964.45.

INDEX TO PUBLIC ROADS, VOLUME 18, NOW AVAILABLE

The index to volume 18 of PUBLIC ROADS is now available. In addition to the index a chronological list of articles and a list of authors are given. The index will be sent free to subscribers to PUBLIC ROADS requesting it. Requests should be addressed to the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C.

Indexes to volumes 6 to 17, inclusive, are also available and will be sent to PUBLIC ROADS subscribers upon request. Indexes to volumes 1 to 5, inclusive, have never been prepared and it is not expected that these volumes will ever be indexed.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF AUGUST 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,357,201	$ 584,800	5.9	$ 8,004,638	$ 3,994,190	335.8	$ 2,217,394	$ 1,106,105	91.5	$ 3,065,095
Arizona	104,551	75,048	6.2	1,660,552	1,258,667	89.3	381,819	243,878	17.4	1,598,412
Arkansas	377,609	375,996	34.9	396,276	395,543	39.6	3,081,724	3,076,222	185.2	1,117,172
California	1,836,771	963,565	65.8	10,413,202	5,459,857	174.5	3,322,421	1,789,424	74.8	1,782,781
Colorado	973,999	534,844	41.7	2,514,863	1,384,056	68.4	1,453,171	807,562	41.1	2,002,388
Connecticut				1,350,040	660,511	14.7	244,930	120,990	2.1	1,536,794
Delaware	54,130	27,065		782,314	385,679	22.2	480,998	241,791	1.4	1,184,277
Florida	384,200	192,100	14.9	3,381,738	1,690,869	70.6	1,768,070	609,435	9.6	2,853,512
Georgia	2,168,089	1,084,044	106.0	6,488,144	3,244,072	309.5	900,870	450,435	50.4	5,518,798
Idaho	1,068,410	636,730	80.7	1,855,713	1,105,107	137.2	216,751	129,578	10.1	1,124,736
Illinois	1,056,000	548,784	32.7	10,548,238	5,268,705	223.0	4,608,150	2,304,074	125.6	2,385,638
Indiana	1,378,350	689,175	38.0	5,316,367	2,652,215	126.0	1,444,800	722,270	26.0	2,537,966
Iowa	1,367,372	628,196	40.4	8,023,334	3,625,683	255.7	1,354,467	635,823	46.7	744,447
Kansas	652,039	326,019	28.9	5,534,428	2,723,421	606.8	4,763,678	2,381,830	344.1	2,865,792
Kentucky	1,221,362	597,939	67.0	4,824,260	2,412,130	134.2	2,788,148	1,394,074	79.0	2,021,125
Louisiana	256,047	128,000	9.3	12,055,777	2,401,505	57.1	1,911,582	853,091	30.7	2,338,123
Maine	1,250,300	625,084	32.6	1,633,946	809,683	33.1	1,062,224	531,111	17.8	284,541
Maryland	308,000	154,000	5.4	1,683,567	840,021	25.4	1,230,411	602,830	18.1	1,992,916
Massachusetts	122,568	61,234	.8	2,676,060	1,338,026	10.0	566,703	283,350	8.6	2,666,105
Michigan	1,729,700	864,850	58.1	7,267,508	3,562,569	141.6	1,736,044	863,117	34.7	1,849,945
Minnesota	539,480	243,094	12.2	5,434,998	2,701,243	293.6	3,276,776	1,630,237	150.3	2,274,026
Mississippi	486,408	216,262	24.2	8,465,880	3,350,131	375.7	1,476,500	390,710	70.2	3,196,842
Missouri	980,863	488,227	37.1	4,948,747	2,429,616	125.4	3,769,827	1,562,684	141.5	3,619,521
Montana	641,438	360,798	28.8	1,107,172	622,419	42.0	506,495	284,901	20.5	4,344,969
Nebraska	1,403,437	701,323	170.2	6,527,170	3,253,984	503.7	2,354,962	543,930	71.2	2,593,404
Nevada	892,813	773,286	79.8	810,627	697,031	101.4	215,060	186,495	4.5	1,437,152
New Hampshire	274,331	135,489	9.2	925,230	461,067	13.8	185,865	92,914	3.0	1,135,339
New Jersey				2,852,361	1,424,918	21.2	211,060	104,010	.5	2,672,543
New Mexico	1,157,176	705,751	145.4	1,423,518	967,881	93.9	704,592	429,729	32.2	779,236
New York	2,119,953	1,030,620	46.7	16,752,376	8,165,917	288.4	2,286,020	1,084,690	26.4	2,950,798
North Carolina	1,725,948	894,015	109.7	6,621,886	3,152,399	294.4	1,923,140	950,440	119.4	2,373,394
North Dakota	2,590	2,590		2,687,171	2,551,245	189.5	830,942	608,893	78.5	3,569,866
Ohio	1,976,430	986,180	26.7	8,242,462	4,092,820	86.4	4,514,690	2,247,018	36.6	6,578,536
Oklahoma	1,371,102	722,664	67.6	5,534,804	2,887,882	176.8	1,642,937	871,146	69.9	2,878,134
Oregon	940,773	524,235	43.6	1,898,758	1,126,214	47.7	352,233	214,990	10.6	2,135,486
Pennsylvania	2,290,471	1,144,186	41.8	6,245,748	3,100,327	90.2	5,993,753	2,993,682	57.7	3,595,333
Rhode Island	202,800	101,400	1.7	939,242	469,621	14.2	301,760	150,880	2.1	1,021,899
South Carolina	1,832,002	819,493	86.1	3,860,502	1,708,331	171.0	1,280,056	589,300	59.4	1,660,984
South Dakota	195,110	115,760	22.9	4,055,762	2,245,790	364.9	1,360,930	751,310	119.1	3,037,872
Tennessee	1,011,340	505,670	37.4	5,056,894	2,528,447	142.3	706,320	353,160	12.4	4,601,818
Texas	4,921,271	2,439,928	318.1	10,518,144	5,215,042	554.4	3,692,287	1,680,518	216.1	7,049,767
Utah	463,180	332,070	67.8	1,275,370	912,270	59.5	668,200	472,350	30.4	1,096,433
Vermont	739,709	324,451	20.3	1,051,840	501,258	28.8	193,250	96,430	4.0	216,361
Virginia	1,368,645	684,322	52.9	5,276,584	2,635,771	174.3	1,920,501	957,721	49.8	760,705
Washington	1,289,826	683,301	26.2	3,349,150	1,753,230	43.4	993,509	521,000	18.4	1,050,896
West Virginia	115,250	92,300	9.2	2,251,333	1,488,565	62.6	784,989	398,438	17.6	2,249,918
Wisconsin	1,878,871	904,877	52.4	6,700,520	3,089,640	192.8	1,685,881	774,190	57.7	1,524,979
Wyoming	584,300	358,877	62.9	1,694,734	1,040,706	215.0	137,200	84,760	18.0	788,145
District of Columbia										
Hawaii	369,705	184,853	6.6	812,825	397,342	15.1	492,600	243,940	5.5	1,139,041
Puerto Rico				1,123,739	562,390	20.5				671,985
TOTALS	49,441,920	25,573,545	2,276.8	224,861,512	110,746,006	7,683.6	79,996,690	40,417,456	2,718.4	114,535,945

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF AUGUST 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 452,600	$ 226,150	30.3	$ 192,005	$ 94,750	13.7	$ 732,072
Arizona	$ 61,871	$ 32,500	3.5	134,185	96,508	7.9	130,629	86,340	9.2	441,134
Arkansas				19,426	12,863					851,245
California	281,757	161,895	20.8	972,288	523,189	54.4	750,036	405,386	22.1	754,463
Colorado	303,280	168,620	17.0	482,446	266,673	29.0	138,040	59,610	9.9	426,665
Connecticut				27,460	13,730	.7	60,470	30,035	.7	274,513
Delaware							64,910	32,455	12.2	214,420
Florida				20,122	10,061					664,791
Georgia	121,120	60,560	16.2	256,858	128,429	33.0	524,497	262,249	64.0	807,484
Idaho	105,529	52,041	8.0	306,582	133,830	32.7	71,984	23,880	13.9	263,821
Illinois	186,000	93,000	21.6	1,664,832	778,416	126.1	832,500	416,250	53.2	739,249
Indiana				780,900	334,750	80.3	978,188	485,850	90.5	421,838
Iowa										1,298,449
Kansas	43,670	21,835	.5	61,294	30,647	.8	250,786	125,392	42.2	1,141,097
Kentucky	309,086	104,551	52.0	524,798	147,811	37.5	1,395,013	385,048	107.8	177,016
Louisiana				36,459	18,135	.8	826,749	343,720	73.6	362,281
Maine	144,600	72,300	9.8	287,726	143,863	17.6	224,400	105,833	9.3	6,577
Maryland				6,263	3,132					409,344
Massachusetts				5,300	2,650		72,710	36,355	1.4	607,395
Michigan				262,966	131,483	24.9	518,500	259,250	36.6	1,144,806
Minnesota	64,876	32,438		344,684	160,286	45.4	142,000	70,670	5.7	1,116,186
Mississippi							317,100	158,500	23.8	730,427
Missouri	189,646	94,630	32.5	246,050	122,635	14.1	557,150	192,825	76.8	654,998
Montana				13,983	7,865					1,027,170
Nebraska	132,830	66,415	17.7	414,998	207,499	64.6	221,782	101,575	44.1	620,563
Nevada	116,610	101,122	11.4	190,090	164,472	30.7	131,727	112,784	44.4	113,834
New Hampshire	97,426	48,277	1.7	111,335	55,301	2.8	39,820	19,880	1.5	123,417
New Jersey				71,490	35,745	1.8	284,710	137,260	3.3	499,768
New Mexico	289,750	176,717	14.4	251,644	153,476	16.3	232,359	132,233	17.2	344,346
New York	699,960	349,980	48.8	1,854,700	911,350	122.5	667,500	321,750	40.1	776,193
North Carolina	29,080	14,540	7.6	910,184	455,070	90.0	213,560	89,720	13.8	522,525
North Dakota				106,900	57,253	26.8	91,100	48,792	16.2	681,207
Ohio				184,400	92,200	3.8	11,100	5,000	.9	1,734,441
Oklahoma				242,508	129,034	16.7	587,110	292,473	45.3	763,408
Oregon	103,929	62,390	15.8	286,593	174,172	31.4	101,414	52,970	13.4	463,590
Pennsylvania	397,444	194,077	28.3	1,325,920	637,768	86.9	1,023,708	511,854	54.0	772,189
Rhode Island	21,730	10,865	.6	113,958	56,979	5.5	56,690	28,345	1.4	91,702
South Carolina				543,081	230,762	61.6	681,685	278,854	69.6	169,064
South Dakota				11,300	6,250					816,436
Tennessee				314,586	157,293	13.1	399,180	138,940	23.4	763,257
Texas	357,714	162,388	73.2	1,810,549	851,769	212.0	1,174,294	448,250	138.1	1,662,362
Utah	70,870	39,550	7.8	339,420	188,770	30.5	175,880	76,960	25.0	245,090
Vermont	163,580	71,150	9.7	84,476	40,703	4.1	76,700	35,450	3.2	62,293
Virginia	108,500	54,250	16.1	648,587	292,883	42.4	384,010	168,304	39.3	376,562
Washington	153,583	80,800	10.6	383,482	201,378	48.2	393,268	207,000	26.6	241,378
West Virginia				244,050	122,025	21.4	82,300	41,150	4.5	386,499
Wisconsin				695,138	331,395	26.7	395,090	191,170	24.2	689,663
Wyoming	290,340	179,400	33.7	149,580	92,420	15.2	235,700	145,650	15.7	167,723
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico				244,000	121,950	13.7				124,925
TOTALS	4,844,781	2,506,291	479.3	18,496,441	9,089,148	1,526.6	15,708,354	7,160,762	1,332.8	28,698,626

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Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

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SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF AUGUST 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			Estimated Total Cost	Federal Aid	NUMBER			
			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise			Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 127,900	$ 127,900	3			$ 451,948	$ 450,924	5			$ 598,910	$ 598,410	6		7	$ 808,285
Arizona						4,718	4,718									625,495
Arkansas						366,437	365,280	8			273,650	273,119	6			1,120,370
California	106,025	106,025		1		1,235,726	1,235,151	4	3		325,621	316,149	3			2,042,884
Colorado	18,855	18,855	1		1	71,588	71,588	1			11,776	8,016			3	1,189,583
Connecticut																844,490
Delaware											77,270	77,270			23	416,480
Florida						10,616	10,616				178,800	178,800	1			1,216,381
Georgia						18,346	18,346				68,200	68,200	3			2,330,841
Idaho	87,825	87,652	1			12,342	12,342				85,392	85,392	3			637,484
Illinois	4,500	4,500			1	1,471,675	1,471,675	6			806,230	806,230	6	1	60	2,922,238
Indiana	269,886	183,900		3		600,600	573,700	5	2		689,780	689,780	1	1	89	966,769
Iowa	521,482	493,000	4		8	496,933	468,200	6	1	6	231,712	216,600	1		7	1,384,766
Kansas	196,494	196,494	6		4	510,094	508,094	6			489,095	489,095	7		4	1,380,825
Kentucky						145,000	145,000	1			515,416	515,416	4	1	31	1,153,629
Louisiana						146,486	146,478	2			327,021	293,050	6			1,137,543
Maine						414,697	414,697	5	2		2,610	2,610			1	273,896
Maryland						64,586	64,586	1								962,247
Massachusetts						70,420	70,420		1		214,320	210,919	1	1		1,631,949
Michigan	110,000	110,000	1	1		886,558	886,558	5		45	566,600	566,600	8	1		1,722,027
Minnesota	40,218	40,218		1	1	777,076	777,076	3	5							1,604,188
Mississippi						355,700	355,700	4								1,228,451
Missouri	66,460	66,460	1	1		285,630	285,630	3			55,560	55,560	1			2,617,603
Montana						642,268	637,386	6			175,365	175,365	2			511,720
Nebraska						325,663	325,663	13			15,264	15,264			7	1,423,483
Nevada	5,626	5,626			2	91,561	91,561		2		57,561	57,561		1	2	303,893
New Hampshire	61,732	61,425		1		7,406	7,406									424,919
New Jersey						210,005	204,779	2	1		15,630	15,630			2	1,749,848
New Mexico						122,441	122,441	4			206,032	206,032	3	1		523,018
New York	130,400	130,400		1		1,929,559	1,923,008	8	5	2	344,963	343,963		2		4,372,558
North Carolina						504,760	504,760	4	2		518,620	463,620	3	6	7	1,476,014
North Dakota						559,858	559,858	1	2		46,420	46,420	1			972,943
Ohio						103,830	103,830	2			195,400	195,400	2			3,929,938
Oklahoma						17,343	17,343				36,075	36,075			13	2,228,091
Oregon	95,263	94,614		2		220,687	220,687	1			9,754	9,754				650,276
Pennsylvania						299,116	283,910	2			200,021	191,000	1			5,257,625
Rhode Island						231,303	231,303	1			159,638	159,638		3		102,809
South Carolina						86,379	63,593	1	2		140,075	140,075			34	1,285,434
South Dakota	3,870	3,870			1	312,558	310,418	2	2	6	114,070	112,670	5		3	891,799
Tennessee						14,381	14,381				87,380	87,380		2	1	1,790,272
Texas	36,660	35,990	2			780,122	780,122	7	1		403,217	402,605	4	2		4,123,072
Utah	1,760	1,760			1	122,518	122,518	3			25,400	25,400	1		3	423,385
Vermont	175,382	170,382	4	1		54,954	54,954	2	1		18,220	18,220			7	236,571
Virginia	120,290	120,290	9	1		289,578	289,578	4	1		164,440	164,440	3		8	1,221,722
Washington	92,233	90,133		2	2	687,331	687,331	8	2		17,162	17,162			2	589,174
West Virginia						412,009	410,989	4			37,470	37,470			5	875,639
Wisconsin	100,586	100,586	1			1,032,228	1,015,047	9			151,644	145,856	3			1,212,020
Wyoming						159,644	159,644		2		14,530	14,530			6	502,556
District of Columbia																325,431
Hawaii						33,230	33,230	1			163,920	163,920	2	1		296,600
Puerto Rico						110,940	110,390	3			116,096	115,440	1			504,470
TOTALS	2,373,447	2,250,080	33	15	21	17,758,848	17,622,909	153	37	59	8,952,330	8,812,106	88	23	325	68,423,704

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 8 OCTOBER 1938

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 8 October 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

THE EFFECT OF USING A BLEND OF PORTLAND AND NATURAL CEMENT ON THE PHYSICAL PROPERTIES OF MORTAR AND CONCRETE [1]

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by **W. F. KELLERMANN**, Associate Materials Engineer, and **D. G. RUNNER**, Assistant Materials Engineer

WITHIN the last few years considerable scaling of concrete pavements has resulted from the use of chemical salts in ice treatment in the Northern States. In an effort to combat this condition the New York State highway department began about 4 years ago to experiment with a blend of portland and natural cement for concrete pavement construction.

Preliminary tests made in the laboratory of the New York Department of Public Works indicated that concretes containing blends of portland and natural cements had greater resistance to alternate freezing and thawing in a solution of calcium chloride than had concretes containing only portland cement. These preliminary laboratory experiments were followed by the construction of a number of experimental sections of actual pavements, using portland cements of various chemical compositions and two natural cements, both made in New York State and both readily available commercially.

The Bureau of Public Roads became interested in this work not only from the technical viewpoint but also because of a request from the Division of Highways of the New York Department of Public Works for permission to conduct certain of its field experiments on pavements financed either wholly or in part with Federal funds. It was felt that before approving the use of blended cements on any extended mileage of highway very definite information should be obtained regarding their effect on other essential qualities, such as strength volume change, etc., and, furthermore, that these tests should be made in the laboratory under closely controlled conditions.

This paper reports the results of such a study as well as the results of a series of freezing and thawing tests on specimens taken from certain of the experimental pavements referred to above. The data on laboratory fabricated specimens are presented in two sections, the first dealing with tests of mortars, and the second with concretes. The results of freezing tests on concrete cores are discussed in a third section.

Tests on the mortar specimens included determinations of crushing strength, flexural strength, absorption, density, durability, and volume change. Two portland cements, differing considerably in chemical composition, and two sands of widely different mineral composition were used in this portion of the investigation. Tests on concrete specimens fabricated in the laboratory included determinations of crushing strength, flexural strength, and modulus of elasticity, using the same two portland cements with local fine and coarse aggregate. One brand of natural cement was used in all specimens fabricated in the laboratory.

The freezing tests discussed in the third part of the report were made on cores taken from four of the experimental concrete pavements. Two natural cements as well as portland cements conforming to four different classifications as to compound composition were used in these roads. Core specimens were taken from sections in which straight portland cement was used as well as from sections containing blends in the proportion of 6 parts of portland cement to 1 part of natural cement, and 5 parts of portland cement to 2 parts of natural cement, by weight. These specimens were subjected to alternate freezing and thawing in water and in a 10 percent solution of commercial calcium chloride.

REPRESENTATIVE MATERIALS USED IN LABORATORY FABRICATED SPECIMENS

The two portland cements included one brand from the Hudson River Valley region, designated as cement A, and one from the Lehigh Valley of Pennsylvania, designated as cement B. Both cements satisfactorily met the requirements for portland cement of the American Society for Testing Materials. The natural cement represented material produced in the northeastern part of the United States and is typical of the old type produced in vertical kilns. The fineness as determined by the Wagner turbidimeter was about the same for both portland cements, but the natural cement had a fineness more nearly corresponding to that of high-early-strength cement. The results of the physical and chemical tests on the three cements are shown in table 1.

Three concrete sands were used in the investigation. Sands A and C were composed essentially of quartz, and sand B consisted essentially of shale and quartz particles with appreciable amounts of limestone, slate, and sandstone. The sieve analyses and physical properties of the three sands and the coarse aggregate used in the tests are given in table 2. The coarse aggregate used in the laboratory fabricated concrete specimens was a limestone of known satisfactory quality and graded uniformly from No. 4- to 1½-inch material.

For the various tests on mortars, each portland cement and blend was used with each sand. Specimens were made using 100 percent portland cement and using two blends, one in which 14 percent and the other in which 28 percent by weight of portland cement was replaced by natural cement. These values correspond to the percentages of natural cement used in the New York experimental roads, which were of the order of 1 sack or 2 sacks in a 7-sack batch, or 14 and 28 percent, respectively.

For each of the portland cements, a sufficient quantity of water was added to the straight portland cement-sand A mixture to give a flow of approximately 90 on

---

[1] Paper presented at the forty-first annual meeting of the American Society for Testing Materials, Atlantic City, N. J., 1938.

Table 1.—*Properties of cements used in laboratory fabricated specimens*

PHYSICAL PROPERTIES

Item	Cement A	Cement B	Natural cement
Apparent specific gravity	3.14	3.11	3.00
Specific surface, cm²/gm	1,730	1,805	2,680
Percent retained on No. 200 sieve	7.1	9.6	10.5
Normal consistency, percent	23.7	24.0	26.0
Time of set, hours:			
Initial	3.3	2.9	1.1
Final	5.1	4.9	1.8
Tensile strength, lb. per sq. in.:			
7 days	380	370	50
28 days	445	445	150

CHEMICAL ANALYSES

Oxides	Percent	Percent	Percent
Silica	22.41	20.80	24.91
Alumina	4.78	6.47	9.65
Iron	3.72	2.38	
Lime	64.55	63.22	33.93
Magnesia	1.15	3.45	19.92
Sulphuric anhydride	1.72	1.77	1.42
Loss on ignition	1.37	1.47	8.55

COMPUTED COMPOUND COMPOSITIONS

Compounds			
Tricalcium silicate	50	47	----------
Dicalcium silicate	26	25	----------
Tricalcium aluminate	6	13	----------
Tetracalcium alumino-ferrite	11	7	----------

Table 2.—*Properties of aggregates*

SIEVE ANALYSES OF FINE AGGREGATES

Total retained on—	Sand A	Sand B	Sand C
⅜-inch sieve	0	0	0
No. 4 sieve	2	4	7
No. 8 sieve	11	21	24
No. 16 sieve	27	45	39
No. 30 sieve	49	67	53
No. 50 sieve	84	87	86
No. 100 sieve	97	95	97
Fineness modulus	2.70	3.19	3.06

PHYSICAL PROPERTIES

Item	Sand A	Sand B	Sand C	Coarse aggregate
Absorption, percent	0.3	2.2	0.9	0.32
Organic matter	(1)	(1)	(1)	----------
Decantation loss, percent	1.4	2.5	1.5	----------
Compressive strength ratio:				
7 days	140	123	121	----------
28 days	130	124	121	----------
Percentage of wear, Deval	----------	----------	----------	3.0

[1] Satisfactory.

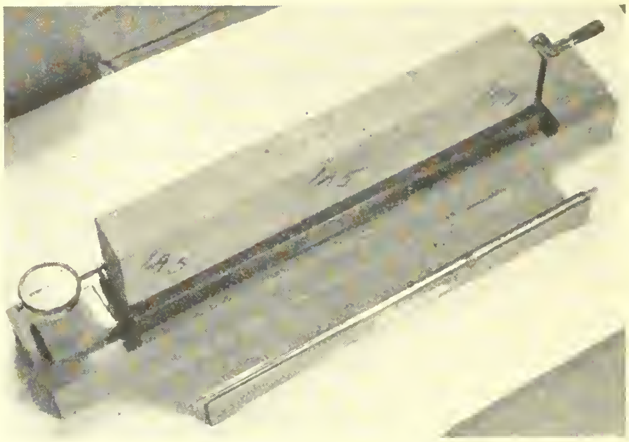

Figure 1.—Horizontal Comparator Used in Measuring Length Change of Mortar Beams.

Table 3.—*Consistencies of mortars as determined by the 10-inch flow table*

MIXTURES CONTAINING SAND A

Percentage of portland cement replaced by natural cement	Cement A		Cement B	
	Beams	Cubes	Beams	Cubes
0	91	93	93	93
14	87	90	90	89
28	83	83	88	85

MIXTURES CONTAINING SAND B

Percentage of portland cement replaced by natural cement	Cement A Beams	Cement A Cubes	Cement B Beams	Cement B Cubes
0	83	83	85	85
14	79	76	82	80
28	74	70	79	78

the 10-inch flow table. The same quantity of water was then used in all other mixtures containing this cement. This resulted in slightly lower flows (drier mortars) for the mixtures containing sand B and also somewhat lower flows for the mixtures containing natural cement. All mixtures, however, were plastic and workable, insofar as could be determined when fabricating the test specimens. The consistencies of the various mixes, as determined by the use of the 10-inch flow table, are shown in table 3.

An outline of the various tests made on the mortar specimens follows.

1. Determinations of crushing and flexural strengths of 2-inch cubes and 2- by 3- by 18-inch beams, using a mix of 1:2 by weight. Specimens were cured continuously in water at 70° F., and broken at ages of 7, 28, 90, 180, and 360 days. Duplicate sets of specimens were cured in air (with 7 days initial water cure) and broken at 28, 90, 180, and 360 days. Beam specimens were broken as cantilevers, using portions of the same beam for tests at various ages.

2. Determinations of tensile strength of briquets containing standard Ottawa sand in a 1:3 mix by weight. Specimens were broken at 7 and 28 days.

3. Tests for volume change on 2- by 3- by 18-inch beams, using a 1:2 mix by weight. One set of specimens was stored in air for 120 days, the other continuously in water at 70° F. for 180 days. As a supplementary portion of this phase of the work, after 180 days of water storage the beams were frozen and thawed for 6 cycles, dried at 120° F. for 4 days, and resaturated in water at 70° F. for 3 days, after which the freezing and thawing cycle was again repeated. In this test measurements of length were made at the conclusion of each 3-day resaturation period. The linear comparator used in all measurements of volume change is shown in figure 1.

4. Miscellaneous tests included absorption and bulk specific gravity tests on 2- by 4-inch cylinders of 1:2 mortar, sodium sulphate soundness tests on 1- by 1- by 6-inch bars, using a mix of 1:5 by weight of Ottawa sand, and autoclave tests on 1- by 1- by 10-inch neat cement bars.

USE OF BLENDED CEMENTS DECREASED STRENGTH OF MORTARS

The results obtained in the strength tests of mortars are given in tables 4 to 7, inclusive, and are shown graphically in figures 2 to 5 inclusive. In the figures, the values have been plotted so as to show on the left the average results for the two sands with each cement and on the right the average results for the two cements with each sand.

Table 4 and figure 2 give the results of crushing tests on specimens continuously water cured. When the results are averaged as in the figure it will be seen that, for each combination of materials, the straight portland cement mixes developed the greatest strengths at all ages. With the exception of the 180- and 360-day values for the straight portland cement mortar in which sand A was used with cements A and B, all mortars exhibited gain in strength from the 7- up to the 360-day periods.

TABLE 4.—*Crushing strengths of 2-inch mortar cubes, mix 1:2 by weight, water cured*

CEMENT A—SAND A

Percentage of portland cement replaced by natural cement	7 days		28 days		90 days		180 days		360 days	
	Lb. per sq. in.[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]
0	6,180	100	8,570	100	9,625	100	9,910	100	9,990	100
14	5,805	94	7,360	86	8,580	89	9,005	91	9,320	93
28	4,620	75	6,105	71	7,255	75	7,595	77	8,290	83

CEMENT A—SAND B

Percentage of portland cement replaced by natural cement	7 days		28 days		90 days		180 days		360 days	
0	5,000	100	6,975	100	8,280	100	9,125	100	9,645	100
14	5,010	100	6,555	94	7,740	94	8,270	91	9,245	96
28	4,430	89	5,845	84	7,190	87	7,880	86	8,660	90

CEMENT B—SAND A

Percentage of portland cement replaced by natural cement	7 days		28 days		90 days		180 days		360 days	
0	6,400	100	8,375	100	9,825	100	9,665	100	9,355	100
14	5,090	80	6,710	80	8,165	83	8,695	90	8,620	92
28	4,130	65	5,420	65	6,920	70	7,175	74	7,490	80

CEMENT B—SAND B

Percentage of portland cement replaced by natural cement	7 days		28 days		90 days		180 days		360 days	
0	5,015	100	6,815	100	7,840	100	8,620	100	8,980	100
14	4,745	95	6,140	90	7,345	94	7,725	90	8,590	96
28	3,930	78	5,215	77	6,645	85	7,120	83	7,735	86

[1] Each value is the average of 15 tests.
[2] Figures indicate percentage of the strength of 100 percent portland cement mixes.

TABLE 5.—*Crushing strengths of 2-inch mortar cubes, mix 1:2 by weight, air cured after 7-day initial water cure*

CEMENT A—SAND A

Percentage of portland cement replaced by natural cement	28 days		90 days		180 days		360 days	
	Lb. per sq. in.[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]
0	8,825	100	8,870	100	8,715	100	9,085	100
14	8,155	92	8,550	96	8,535	98	8,865	98
28	6,895	78	7,335	83	7,540	87	7,870	87

CEMENT A—SAND B

Percentage of portland cement replaced by natural cement	28 days		90 days		180 days		360 days	
0	7,205	100	7,555	100	7,870	100	7,105	100
14	7,170	100	7,470	99	7,780	99	6,885	97
28	6,540	91	7,100	94	7,210	92	6,295	89

CEMENT B—SAND A

Percentage of portland cement replaced by natural cement	28 days		90 days		180 days		360 days	
0	8,755	100	8,975	100	9,040	100	8,765	100
14	7,530	86	7,980	89	8,255	91	8,175	93
28	6,290	72	6,705	75	7,065	78	6,920	79

CEMENT B—SAND B

Percentage of portland cement replaced by natural cement	28 days		90 days		180 days		360 days	
0	7,040	100	7,425	100	7,670	100	7,195	100
14	6,700	95	6,895	93	7,465	97	6,525	91
28	5,495	78	5,505	74	5,825	76	6,040	84

[1] Each value is the average of 15 tests.
[2] Figures indicate percentage of the strength of 100 percent portland cement mixes.

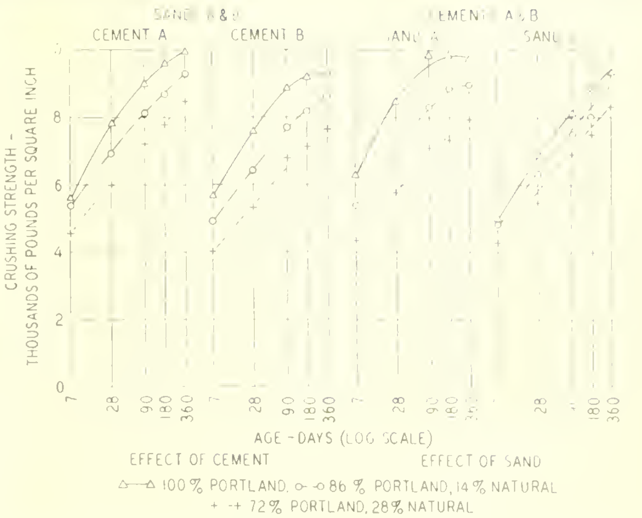

FIGURE 2.—EFFECT OF BLENDING PORTLAND AND NATURAL CEMENT ON THE CRUSHING STRENGTH OF 2-INCH MORTAR CUBES CURED IN WATER.

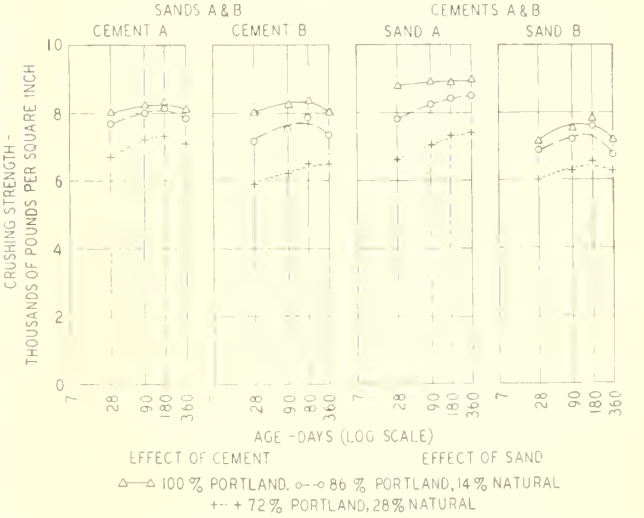

FIGURE 3.—EFFECT OF BLENDING PORTLAND AND NATURAL CEMENT ON THE CRUSHING STRENGTH OF 2-INCH MORTAR CUBES CURED IN AIR.

In the left hand portion of figure 2 it may be seen that mortars containing cement A had slightly greater strengths than those containing cement B. The right hand portion of this figure shows the effect of using different sands. In the unblended mix, as well as in the mortar containing 14 percent of natural cement, specimens containing sand A showed greater strengths at all ages. However, in the mixes containing 28 percent of natural cement, the strengths were about the same. In other words, replacing portland cement with natural cement seemed to affect the strength of mortars made with sand A much more than those made with sand B.

The values obtained on air-cured, 2-inch cubes (after 7 days of initial water storage) are given in table 5 and in figure 3. The average values given in the figures

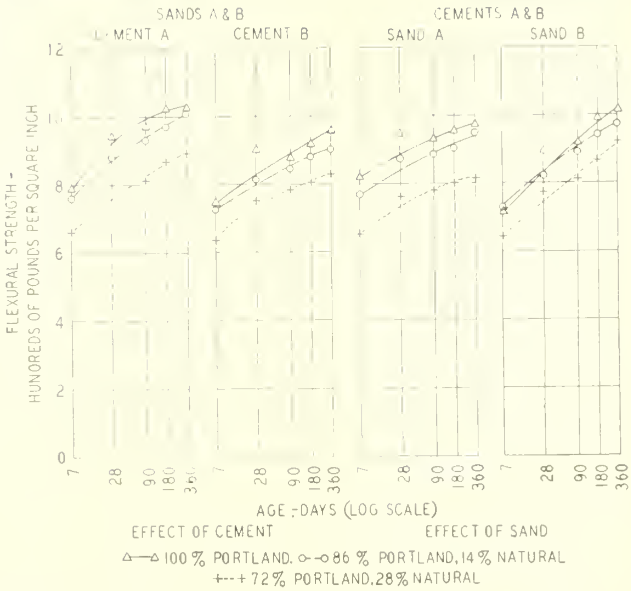

FIGURE 4.—EFFECT OF BLENDING PORTLAND AND NATURAL CEMENT ON FLEXURAL STRENGTH OF 18-INCH MORTAR BEAMS CURED IN WATER.

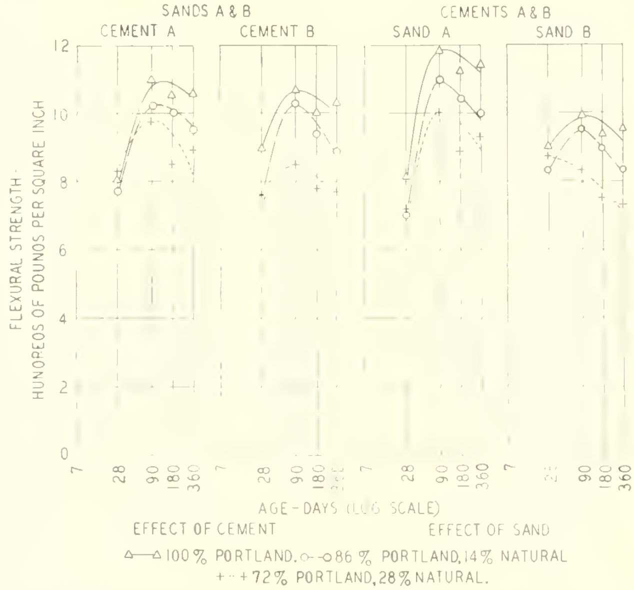

FIGURE 5.—EFFECT OF BLENDING PORTLAND AND NATURAL CEMENT ON FLEXURAL STRENGTH OF 18-INCH MORTAR BEAMS CURED IN AIR.

show, in all cases, higher strengths for the mixes containing straight portland cement. Comparing figures 2 and 3 it will be noted that the specimens moist-cured 7 days and air-dried for 21 days were in general higher in strength than those cured continuously in water for 28 days. Further air storage, however, resulted in only slight gain in strength at 90 days, with little or no gain thereafter.

TABLE 6.—*Flexural strengths of 18-inch mortar beams, mix 1:2 by weight, water cured*

Percentage of portland cement replaced by natural cement	7 days		28 days		90 days		180 days		360 days	
	Lb. per sq. in.[1]	*Per-cent*[2]	*Lb. per sq. in.*[1]	*Per-cent*[2]	*Lb. per sq. in.*[1]	*Per-cent*[2]	*Lb. per sq. in.*[1]	*Per-cent*[2]	*Lb. per sq. in.*[1]	*Per-cent*[2]
CEMENT A—SAND A										
0	850	100	995	100	970	100	1,010	100	1,030	100
14	770	91	905	91	940	97	945	94	995	97
28	660	78	780	78	795	82	825	82	880	85
CEMENT A—SAND B										
0	735	100	900	100	965	100	1,045	100	1,025	100
14	755	103	860	96	920	95	985	94	1,055	103
28	660	90	820	91	835	87	915	88	915	89
CEMENT B—SAND A										
0	790	100	905	100	885	100	900	100	915	100
14	760	96	845	93	835	94	865	96	915	100
28	650	82	745	82	765	86	790	88	760	83
CEMENT B—SAND B										
0	700	100	890	100	870	100	930	100	1,000	100
14	700	100	795	89	865	99	895	96	885	89
28	625	89	745	84	800	92	820	88	910	91

[1] Each value is the average of 5 tests.
[2] Figures indicate percentage of the strength of 100 percent portland cement mixes.

TABLE 7.—*Flexural strengths of 18-inch mortar beams, mix 1:2 by weight, air cured*

Percentage of portland cement replaced by natural cement	28 days		90 days		180 days		360 days	
	Lb per sq. in.[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]
CEMENT A—SAND A								
0	720	100	1,195	100	1,140	100	1,180	100
14	680	94	1,110	93	1,070	94	1,035	88
28	725	100	1,085	91	915	80	970	82
CEMENT A—SAND B								
0	885	100	1,005	100	950	100	935	100
14	865	98	930	93	920	97	875	94
28	930	105	850	85	795	84	805	86
CEMENT B—SAND A								
0	905	100	1,165	100	1,095	100	1,100	100
14	700	77	1,080	93	1,015	93	970	88
28	710	79	905	78	855	78	880	80
CEMENT B—SAND B								
0	900	100	970	100	915	100	975	100
14	800	89	970	100	860	94	800	82
28	810	90	810	83	705	77	655	67

[1] Each value is the average of 5 tests.
[2] Figures indicate percentage of the strength of the 100 percent portland cement mixes.

Figure 3 shows the effect of the cement and sand upon the crushing strength of the air-dried specimens. Specimens containing cement A exhibited slightly greater strengths in the blends, with the unblended mortars having about the same strength at all ages. Specimens containing sand A showed definitely higher strengths throughout. The most interesting point in connection with this figure is the decrease in strength caused by the replacement of 28 percent of the portland cement with natural cement. The strengths obtained with this combination ran about 1,000 to 2,000 pounds per square inch lower than those of the straight portland cement mixes for the different cements and sands.

The effect of blending portland and natural cement upon the flexural strength of water-cured mortar beams is shown in table 6 and in figure 4. It will be noted that when the results are averaged as in figure 4, for

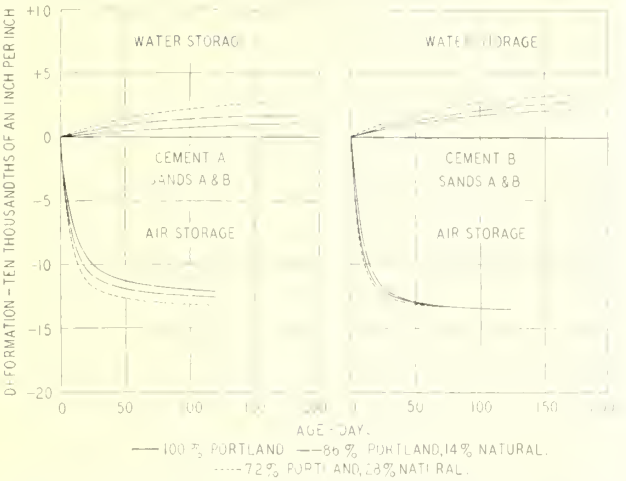

Figure 6.—Effect of Blending Portland and Natural Cement on Volume Change of 18-Inch Mortar Beams.

each combination of materials, the straight portland cement mixes, with a single exception, produced the greatest strengths at all ages. As in the water-cured compression specimens, the gain in strength with age was fairly uniform. However, the reduction in strength caused by blending was not proportional to the amount of natural cement used, the 28-percent replacement causing a proportionately greater reduction than the 14-percent replacement. Cement A in general produced greater strengths for all combinations of materials than did cement B.

The results of flexure tests on air-cured beams are given in table 7 and in figure 5. It will be observed that here also for each combination of materials, with a single exception, the straight portland cement mixes produced the greatest strengths at all ages. With few exceptions the highest strengths were obtained at 90 days, all mixes showing retrogression after that period. In every case the strengths at 90 days were higher than those of the corresponding water-cured specimens.

The effect of blending portland and natural cement upon volume change of 18-inch mortar beams in continuous water storage is shown graphically in the upper portion of figures 6 and 7. The changes are shown in 0.0001 inch per inch at the ages indicated on the abscissas. It will be noted from the figures that blends containing 28 percent of natural cement had the greatest expansion at 180 days, and that the straight portland cement mixes for all combinations exhibited the least expansion. It is of interest to note (fig. 6) that specimens containing cement B had slightly greater expansions for both blended and unblended mixtures than those containing cement A. However, as will be seen from figure 7, changing the sand had very little effect upon volume change.

MORTAR BEAMS SUBJECTED TO ACCELERATED WEATHERING TESTS

Volume changes resulting from continuous air storage are shown in the lower portions of figures 6 and 7. The treatment of these beams consisted of storage for 24 hours in the moist closet, at the end of which time an initial reading was taken, followed by continued storage in laboratory air. During the period from 90 to 120 days the beams were placed in a cabinet in which the air was dried with calcium chloride to decrease further the amount of moisture in the specimens. As expected, all beams showed slightly increased shrinkage during this period.

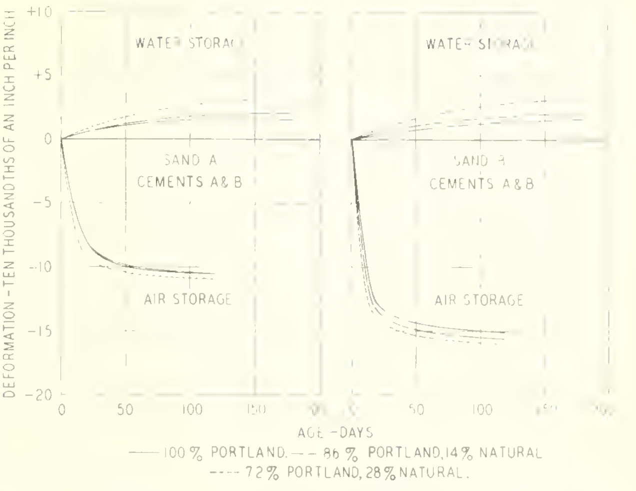

Figure 7.—Effect of Blending Portland and Natural Cement on Volume Change of 18-Inch Mortar Beams.

Beams containing sands A and B and cement A had approximately the same amount of shrinkage as those containing cement B (fig. 6). However, specimens containing cements A and B and sand B had shrinkages at 120 days about 50 percent greater than those containing sand A (fig. 7). The figures show that although the initial shrinkage was somewhat greater in specimens containing the blended cements, the total shrinkage at the end of 120 days of continuous air storage was about the same.

The mortar beams from the continuous water storage series were measured for length at the end of 180 days, and then subjected to 15 rounds of the following accelerated weathering treatment. Each round required 13 days for completion and consisted of 6 cycles of freezing and thawing, each 24 hours in duration, followed by drying for 4 days in an electric oven at 120° F., and resaturation for 3 days in water at 70° F. Readings were taken on each beam at the end of the resaturation period and were discontinued at the end of the fifteenth round, at which time surface disintegration of the specimens had progressed to a point which affected the accuracy of the readings because of loosening of the gage points. Unit length changes at the end of the first, third, sixth, ninth, twelfth, and fifteenth rounds are shown in figures 8 and 9. Here the data have been averaged in the same manner as in the previous figures so as to bring out the effect of variations in the cement and sand.

Several points in connection with these figures are of interest. In the first place, all specimens showed an initial contraction followed by expansion up to the fifteenth round. The amount of initial contraction seemed to have been affected greatly by the combination of materials used, being greatest for the combinations involving cement A with the two sands (fig. 8) and sand B with the two cements (fig. 9). The combinations involving sand A with cements A and B showed very little initial contraction. This effect may have resulted from the drying cycle which was introduced. Drying at 120° F. for 4 days undoubtedly induced high shrinkage in the cement paste, subjecting the particles of aggregate to high compressive stress. It is possible that the difference in the ability of the sands to resist this stress without permanent deforma-

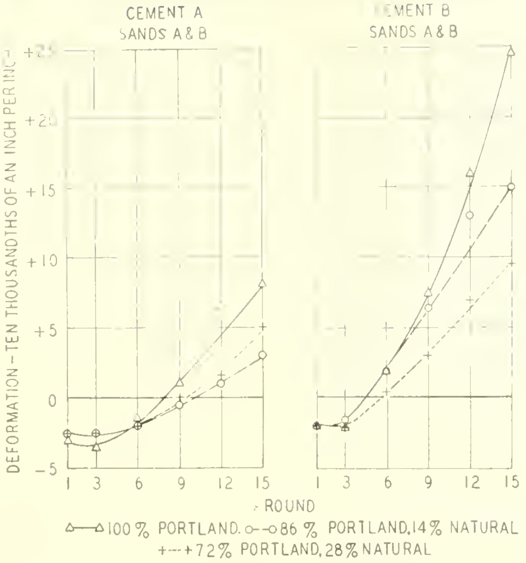

Figure 8.—Effect of Freezing and Thawing, Followed by Drying, Cooling, and Resaturation on Volume Change of Mortar Beams.

tion may have accounted for the residual shrinkage observed in the specimens containing sand B, which was composed of comparatively low-strength material.

The quartz particles composing sand A may have possessed sufficient strength to resist the shrinkage stresses without permanent deformation, so that the specimens when resaturated returned practically to their original length. The accumulated effect of this action may account also for the relatively small expansions which were observed for specimens containing sand B as compared to the expansions shown by specimens containing sand A.

It will be observed also that, without exception, specimens containing the blended cement showed lower expansions than the corresponding specimens containing only portland cement. It is usually assumed, insofar as expansion is concerned, that repeated freezing and thawing produces a gradually expanding paste, resulting in the eventual disintegration of the specimen. On this basis, the use of the blended cements may be considered beneficial. However, visual examination of the specimens as the test proceeded failed to reveal any consistent difference in appearance at any stage between the specimens containing the blended cements and those containing the straight portland cement. So far as could be determined by visual inspection of the surface, all of the specimens failed at about the same rate. All were badly exfoliated or surface pitted by the end of the fifteenth round and apparently possessed little strength.

BLENDING INCREASED RESISTANCE OF MORTARS TO SULPHATE ACTION

The effect of replacements with natural cement on the tensile strength of 1:3 Ottawa sand briquets is shown in table 8. It is of interest to note that, for both cements at 28 days, the specimens containing 14 percent of natural cement had very nearly the same strengths as the straight portland cement mortars. The strengths at 28 days of specimens containing 28 percent of natural cement were approximately 90 percent of the strengths of the straight portland cement mortars.

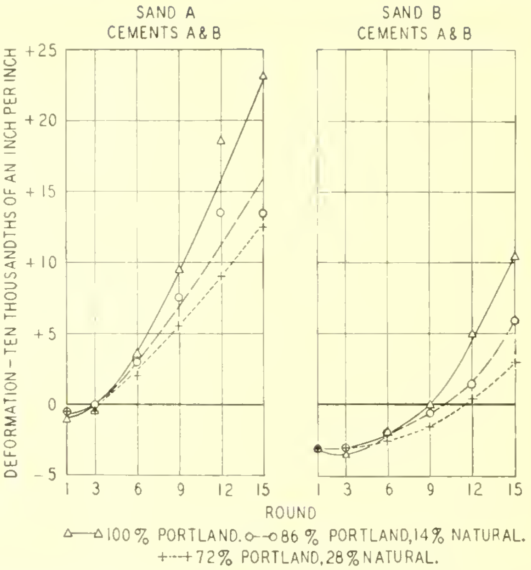

Figure 9.—Effect of Freezing and Thawing, Followed by Drying, Cooling, and Resaturation on Volume Change of Mortar Beams.

Table 8.—*Tensile strengths of briquets, mix 1:3, standard Ottawa sand*

Percentage of portland cement replaced by natural cement	Cement A				Cement B			
	7 days		28 days		7 days		28 days	
	Lb. per sq. in.[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]	*Lb. per sq. in.*[1]	*Percent*[2]
0	390	100	455	100	390	100	450	100
14	370	95	450	99	360	92	440	98
28	345	88	405	89	330	85	405	90

[1] Each value is the average of 15 tests.
[2] Figures indicate percentage of the strength of 100 percent portland cement mixes.

The effect of blending the cements upon the bulk specific gravity and absorption of mortar specimens is shown in table 9. Cylinders 2 inches by 4 inches in size of 1:2 mortar, after 24 hours in moist air, were stored in water at 70° F. and the bulk specific gravity and absorption determined after 7 and 28 days by first determining the saturated-surface dry weights of the specimens in air and in water and then drying to constant weight. It will be observed that blending with natural cement lowered the bulk specific gravity and increased the absorption in all cases. The decrease in density which is indicated by these tests may have been the result of a slight increase in the air voids brought about by the use of natural cement. Other tests have indicated that air voids in concrete are increased somewhat by using blends.

Bars 1 by 1 by 6 inches long, using a 1:5 standard Ottawa sand mortar, and with 0, 14, and 28 percent of the portland cement replaced with natural cement, were fabricated and stored for 6 days in water at 70° F. At the end of this period the bars were measured for length and then immersed in a 10-percent solution of sodium sulphate. After 21 days of continuous storage in this solution at 70° F., they were again measured for length. The amount of expansion was read to 0.001 inch. The results are shown in figure 10 (left panel). Each value is the average of three tests. It will be noted that specimens containing cement B showed greater expansion in all cases than those con-

TABLE 9.—*Bulk specific gravity and absorption of 2 by 4-inch mortar cylinders, mix 1:2 by weight, water cured*

Percentage of portland cement replaced by natural cement	Bulk specific gravity [1]				Percentage absorption [1]			
	7 days		28 days		7 days		28 days	
		Percent [2]		*Percent* [2]		*Percent* [2]		*Percent* [2]
CEMENT A—SAND A								
0	2.10	100	2.12	100	9.52	100	9.23	100
14	2.07	99	2.09	99	9.89	104	10.01	108
28	2.03	97	2.04	96	10.32	108	10.59	115
CEMENT A—SAND B								
0	2.09	100	2.10	100	10.49	100	10.32	100
14	2.07	99	2.08	99	10.84	103	10.73	104
28	2.04	98	2.05	98	11.34	108	11.19	108
CEMENT B—SAND A								
0	2.07	100	2.11	100	10.03	100	9.98	100
14	2.05	99	2.06	98	10.66	106	10.62	106
28	2.02	98	2.02	96	11.27	112	11.40	114
CEMENT B—SAND B								
0	2.09	100	2.07	100	10.95	100	10.76	100
14	2.04	98	2.06	100	11.34	104	11.11	103
28	2.01	96	2.04	99	11.73	107	11.57	108

[1] Each value is the average of 3 tests.

[2] Figures indicate percentage of the corresponding values for 100 percent portland cement mixes.

taining cement A, and also that blending with natural cement reduced the amount of expansion in both cases. These tests seem to indicate that blending with natural cement increases resistance to the action of sodium sulphate solutions.

The expansions of neat cement bars exposed for 3 hours at 420° F. in the autoclave are shown in the right panel of figure 10. In both cases, the replacement of 14 and 28 percent of portland cement by natural cement resulted in increased expansion. It will be noted, however, that cement B, unblended, showed considerably greater expansion than cement A with a 28 percent replacement of natural cement. The relatively high expansions shown by cement B may possibly be associated with the higher magnesia and greater C_3A content of this cement. (See table 1.)

The same two portland cements and the same natural cement that were used in the mortar tests were combined with a local fine and coarse aggregate in fabricating the concrete specimens. The characteristics of the aggregates are shown in table 2. Sand C was used in the concrete tests. The coarse aggregate was uniformly graded from No. 4 to 1½-inch. Two blends were used, one with 14 percent and the other with 28 percent of natural cement as in the mortar tests.

A typical concrete paving mix containing 6 sacks of cement per cubic yard with sufficient water for a 2½-inch slump was used. This resulted in a net water-cement ratio by volume of approximately 0.75.

Although the tests made on the mortars showed that the consistency of the mix was affected somewhat by the substitution of natural cement for portland cement, the slump of the concrete containing the blends was the same as that obtained with the straight portland cement. However, in manipulating the concrete it was found that the blended cement mixtures had a stickiness which was not apparent in mixtures containing the straight portland cement.

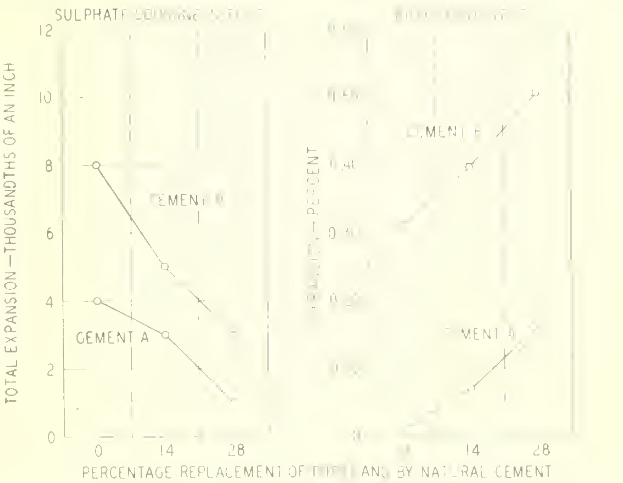

FIGURE 10.—EFFECT OF BLENDING PORTLAND AND NATURAL CEMENT UPON VOLUME CHANGE IN SULPHATE SOLUTION AND IN THE AUTOCLAVE.

USE OF BLENDED CEMENTS SLIGHTLY DECREASED STRENGTH OF CONCRETE

Flexure tests were made on beams 6 by 6 by 21 inches in size, loaded at the third points of an 18-inch span. Compression tests as well as measurements for modulus of elasticity, using a Martens extensometer, were made on 6-inch by 12-inch cylinders. Five specimens were made for each test condition and tests were made at 7, 28, 90, 180, and 360 days. All specimens were stored continuously in moist air at 70° F. until tested.

The results of the tests for strength and modulus of elasticity on laboratory fabricated concrete specimens are shown in table 10 and are shown graphically in figure 11.

Referring to the data for cement A (table 10 and left panel of fig. 11), it will be observed that there was a gradual increase in crushing strength with age for specimens containing the unblended as well as the blended cement. Blending with natural cement decreased the average strength at all ages about 1 percent in the specimens containing 14 percent of natural cement and about 8 percent in the specimens containing 28 percent of natural cement. Similar data for cement B are shown in the next panel to the right. In the specimens containing straight portland cement it will be observed that the same order of strengths was obtained as with cement A. However, in specimens with the blended cements as compared to the specimens with straight portland cement, the reduction in strength was more marked, the corresponding figures being 7 percent for the 14 percent blend and 17 percent for the 28 percent blend.

It is interesting to observe that specimens containing the 14 percent blend for both cements A and B exhibited higher strengths at 360 days than the corresponding straight portland cement specimens. This was due to the fact that the straight portland cement concretes showed no increase in strength beyond the 180-day period for cement A, while for cement B no increase was observed after the 90-day period. On the other hand, the concrete containing the blended cements showed a fairly uniform increase in strength up to 360 days.

Flexural strengths of the concrete specimens are shown also in table 10 and are plotted in the two center panels of figure 11. For each cement the specimens containing the 14 percent blend had strengths com-

TABLE 10.—*Effect of blending portland and natural cements on crushing and flexural strengths and modulus of elasticity*

Percentage of portland cement replaced by natural cement	Test results at age indicated[1]									
	7 days		28 days		90 days		180 days		360 days	
CRUSHING STRENGTH—CEMENT A										
	Lb. per sq. in	*Percent*[2]	*Lb. per sq. in.*	*Percent*[2]	*Lb. per sq. in.*	*Percent*[2]	*Lb. per sq. in.*	*Percent*[2]	*Lb. per sq. in.*	*Percent*[2]
0	3,730	100	5,510	100	6,190	100	6,650	100	6,660	100
14	3,860	103	5,270	96	6,010	97	6,470	97	6,830	103
28	3,430	92	4,750	86	5,870	95	6,080	91	6,480	97
MODULUS OF RUPTURE—CEMENT A										
0	600	100	683	100	777	100	858	100	831	100
14	612	102	697	102	777	100	845	98	857	103
28	542	90	631	92	742	95	801	93	853	103
MODULUS OF ELASTICITY—CEMENT A										
	1,000 lb. per sq. in.		*1,000 lb. per sq. in.*		*1,000 lb. per sq. in.*		*1,000 lb. per sq. in*		*1,000 lb. per sq. in.*	
0	5,272	100	6,047	100	6,490	100	6,785	100	7,102	100
14	5,170	98	5,947	98	6,670	103	6,735	99	7,432	105
28	5,312	101	5,557	92	6,268	97	6,285	93	6,628	93
CRUSHING STRENGTH—CEMENT B										
	Lb. per sq. in.		*Lb. per sq. in.*		*Lb. per sq. in.*		*Lb. per sq. in.*		*Lb. per sq. in.*	
0	3,990	100	5,640	100	6,450	100	6,310	100	6,480	100
14	3,700	93	5,000	89	5,760	89	5,960	94	6,530	101
28	3,190	80	4,330	77	5,250	81	5,670	90	5,730	88
MODULUS OF RUPTURE—CEMENT B										
0	620	100	677	100	781	100	783	100	821	100
14	577	93	698	103	753	96	820	105	859	105
28	542	87	580	86	716	92	801	102	788	96
MODULUS OF ELASTICITY—CEMENT B										
	1,000 lb. per sq. in.		*1,000 lb. per sq. in.*		*1,000 lb. per sq. in.*		*1,000 lb. per sq. in.*		*1,000 lb. per sq. in.*	
0	5,225	100	6,083	100	6,420	100	6,638	100	6,970	100
14	5,065	97	5,810	96	6,178	96	6,465	97	7,070	101
28	5,030	96	5,623	92	6,018	94	6,330	95	6,408	92

[1] Each value for crushing strength and modulus of rupture is the average of 5 tests. Each value for modulus of elasticity is the average of 4 tests.

[2] Figures indicate percentage of the strength of 100 percent portland cement mixes.

parable to those obtained with the straight portland cement specimens. Specimens with the 28 percent blend had strengths that averaged about 95 percent of those of the straight portland cement specimens. It is to be observed also that for both portland cements the blended cement concrete showed higher strength ratios in flexure than in compression.

Results of tests for modulus of elasticity are shown in the two right panels of figure 11. These results are also included in table 10. Here again the addition of the natural cement is reflected in the somewhat lower values obtained with specimens containing the blended cements.

In general, it may be said that the values obtained in flexure and modulus of elasticity for the specimens containing the 28 percent blend were about 95 percent of those obtained for the specimens containing the straight portland cements. The strength ratios of specimens in compression were not the same for both cements, specimens containing cement B giving an average ratio of 83 percent, which was about 10 percent lower than that obtained for specimens containing cement A. Comparing the results of tests on concrete specimens with the corresponding results of tests on mortar specimens, it will be observed that replacement with the natural cement had a much greater effect on mortar strength than on concrete strength. This would, of course, be expected because of the higher cement content in the mortar specimens.

FREEZING AND THAWING TESTS MADE ON PAVEMENT CORES

The results of freezing and thawing tests on cores taken from four experimental projects in New York State are given in this portion of the report. On each of these projects specimens of straight portland cement concrete and blended cement concrete were taken from sections containing cement conforming to each of the four classifications as to compound composition given in table 11.

Natural cement of the brand used in the laboratory tests already described was used on two of the projects. The other two contained natural cement from another mill in New York State. A 6:1 blend was used on two of the projects, a 5:2 blend on the other two.

TABLE 11.—*Classification of four cements as to compound composition*

Specification designation	Tricalcium aluminate C_3A (percent)	Tricalcium silicate C_3S (percent)
935	Not over 9	Not over 35.
950	Not over 9	Between 48 and 52.
1439	Not less than 14	Not over 39.
1450	Not less than 14	Between 48 and 52.

This portion of the work was done for the purpose of testing specimens of concrete which had been subjected to the usual field manipulation during placing in order to study the effects of the "bleeding" or "water-gain" which frequently occurs during the finishing operation. This action results in the accumulation of an excess amount of water in the top surface of the pavement, leaving a weak, porous layer which is lacking in resistance to the disintegrating effects of ice and chemical salts.

In a preliminary series of freezing and thawing tests made on full depth cores it was found that disintegration was confined almost entirely to the top 2 inches of the specimen. Because of this, and also to facilitate performance of the test, it was decided to study the effect of the blend by testing the upper portion of the concrete only. Disks 2 inches thick were sawed from the tops of the cores. One set of these disks was alternately frozen and thawed in a 10 percent solution of calcium chloride, the other set in plain water. In addition, a few tests were made to determine the resistance of similar disks sawed from the lower surface of the cores.

It was found desirable in this work to use a minimum amount of the chloride solution and to freeze rapidly in order to keep the solution from separating into layers of different density. This was accomplished by using close-fitting containers and immersing these containers in the brine solution of the refrigerator. The minimum temperature obtained was approximately −10° F., one complete cycle of freezing and thawing being performed each day. The time required to lower the temperature of the specimen to this minimum was approximately 10 hours. The calcium chloride solution was changed after 10 cycles of freezing and thawing. These time-temperature relations applied to freezing both in chloride and in water.

Periodic examinations were made of the specimens, this examination consisting of weight loss determina-

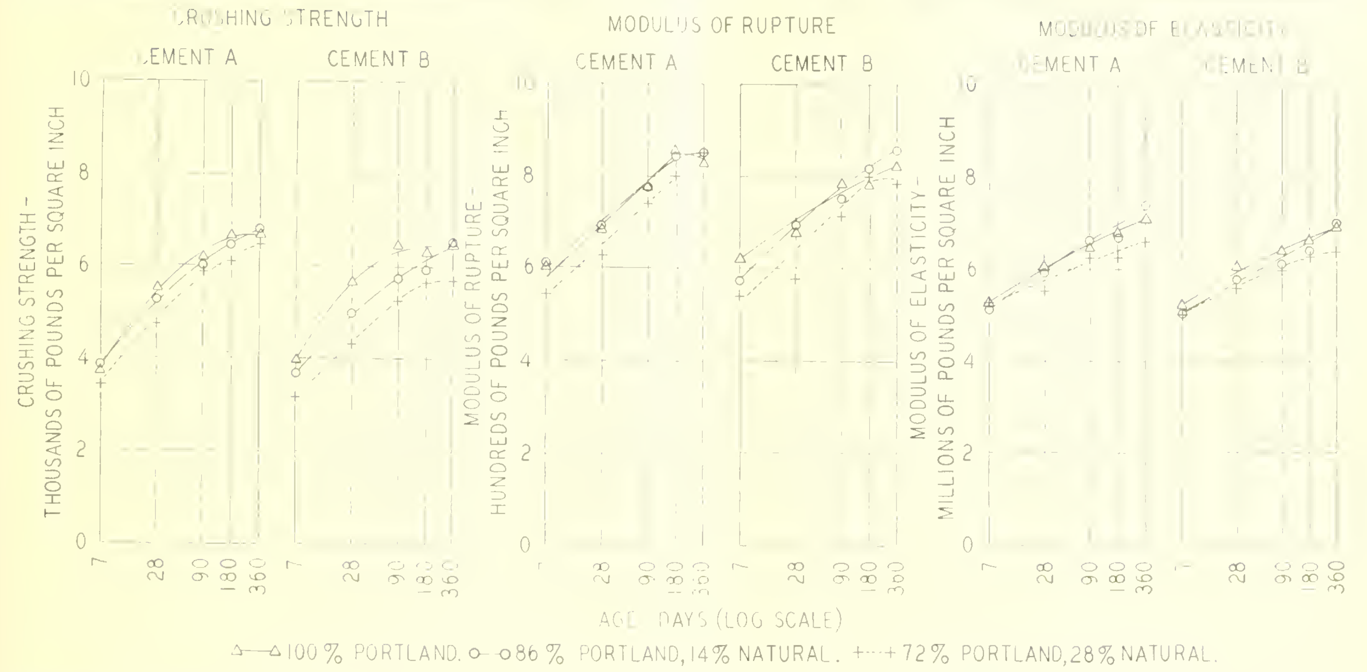

FIGURE 11.—EFFECT OF BLENDING PORTLAND AND NATURAL CEMENT ON STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE.

tions and visual inspections. It was found that the weight losses of the 2-inch disks were meaningless. For instance, a specimen which had not lost more than 2 percent in weight would become unsound and disintegrate. While some scaling of the surface developed, unsoundness in general was caused by failure of the bond between the cement and aggregate, resulting in the eventual disintegration of the entire specimen.

For each project and for each type of freezing there were two groups of 8 specimens each. In one group there were 4 pairs of specimens containing straight portland cement and 4 pairs containing the same portland cements blended with natural cement in the proportion of either 6:1 or 5:2. The 4 pairs of specimens in each group represented concrete containing the 4 classes of portland cement mentioned above.

In order to evaluate the resistance to freezing and thawing, the individual specimens were examined at various intervals and rated in accordance with their appearance and ring when struck with a hammer. A specimen which was entirely sound was given a rating of 12½. An unsound specimen was rated at 2½, while a specimen which appeared to be between these two conditions was considered as questionable and was given a rating of 7½. Any specimen which had disintegrated was given a rating of zero. If all of the specimens of a particular group were sound the rating would be 100. On the other hand, if all of the specimens were disintegrated, the rating would be zero. By this method of rating, the results obtained with the four cements used on each project were averaged for comparison with the results obtained with the same portland cements blended with natural cement. This procedure was decided upon after a study of the individual results had failed to reveal any consistent relation between the composition of the portland cement and its resistance to freezing and thawing either when used straight or when blended. With a few exceptions, concrete made from portland cements conforming to the various composition classifications, when used on a given job, behaved about alike insofar as resistance to freezing and thawing was concerned.

BLENDED SPECIMENS MOST RESISTANT TO FREEZING IN CALCIUM CHLORIDE SOLUTION AND WATER

The ratings of the several groups of specimens after 35 alternations in a 10-percent solution of calcium chloride as well as the corresponding ratings when frozen in water for various periods up to 150 alternations, are given in table 12. The appearance of each individual specimen at the end of 35 alternations in calcium

TABLE 12.—*Resistance of core specimens to freezing and thawing*

Blend [1]	Rating [2] at number of freezing and thawing cycles indicated					
	35 cycles in calcium chloride	35 cycles in water	50 cycles in water	75 cycles in water	100 cycles in water	150 cycles in water
PROJECT A						
7-O	0	45	10	0	0	0
6-1G	90	95	90	90	90	90
PROJECT B						
7-O	25	50	30	25	25	25
5-2G	100	100	100	100	100	100
PROJECT C						
7-O	40	30	5	[3] 0	0	0
6-1H	15	25	10	[4] 0	0	0
PROJECT D						
7-O	0	10	5	0	0	0
5-2H	55	65	55	30	30	25

[1] Figures indicate the number of sacks of portland and natural cement in a 7-sack batch; letters refer to the brand of natural cement.

[2] Rating of 100 indicates all 8 specimens sound; rating of 0 indicates all 8 specimens disintegrated.

[3] At 65 alternations.

[4] At 60 alternations.

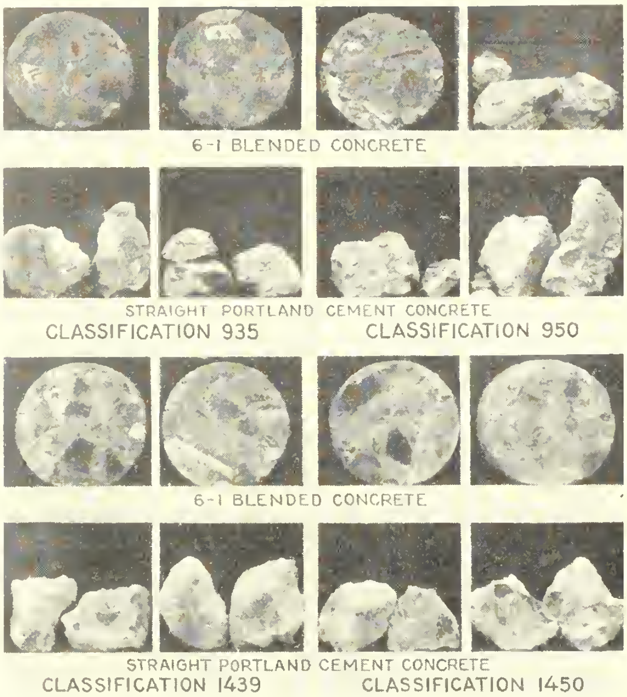

Figure 12.—Project A. Appearance of Blended and Straight Portland Cement Concrete Specimens after 35 Alternations of Freezing and Thawing in a 10 Percent Solution of Calcium Chloride.

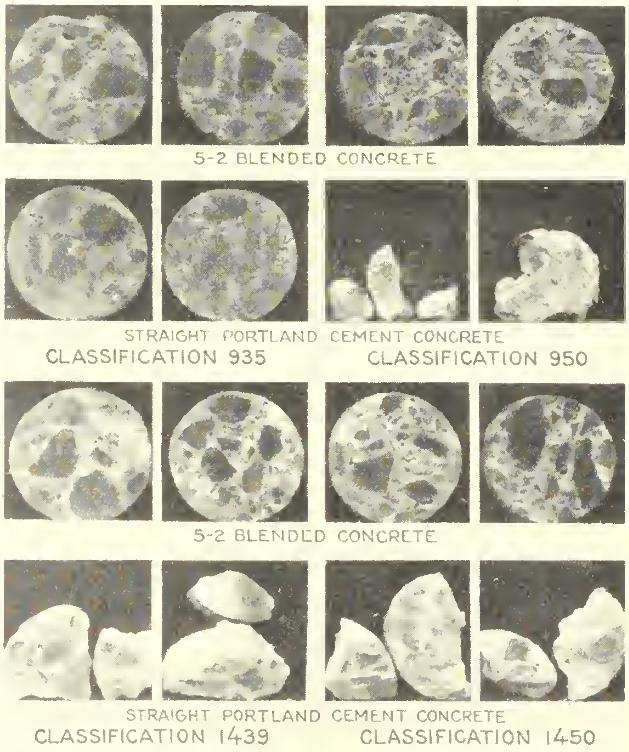

Figure 13.—Project B. Appearance of Blended and Straight Portland Cement Concrete Specimens after 35 Alternations of Freezing and Thawing in a 10 Percent Solution of Calcium Chloride.

chloride and at the end of 50 alternations in water is shown in figures 12 to 19 inclusive. In each of these figures the specimens are arranged as follows: The four specimens shown in the upper left quadrant contain

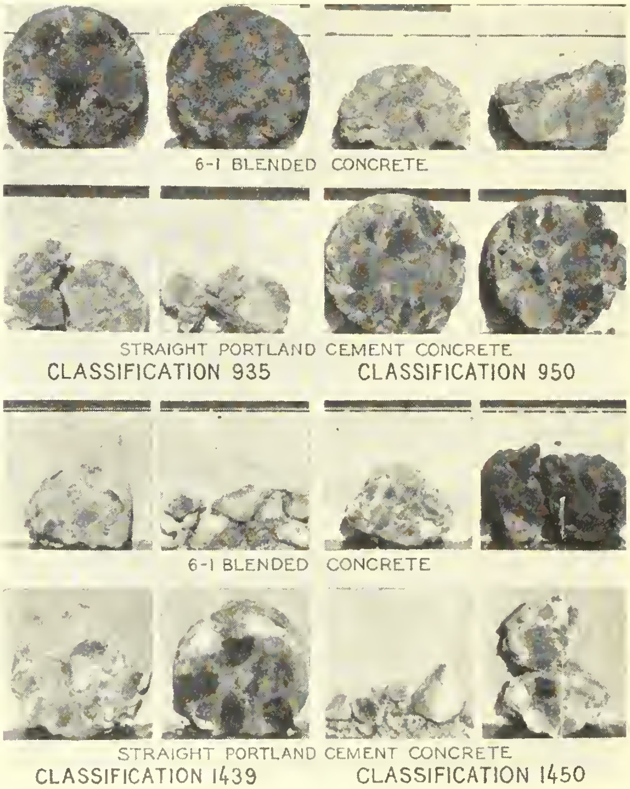

Figure 14.—Project C. Appearance of Blended and Straight Portland Cement Concrete Specimens after 35 Alternations of Freezing and Thawing in a 10 Percent Solution of Calcium Chloride.

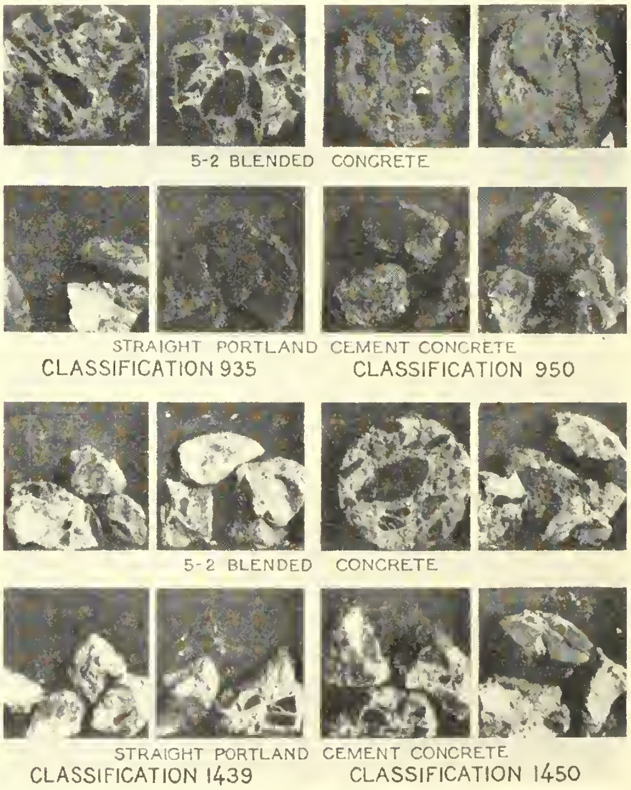

Figure 15.—Project D. Appearance of Blended and Straight Portland Cement Concrete Specimens after 35 Alternations of Freezing and Thawing in a 10 Percent Solution of Calcium Chloride.

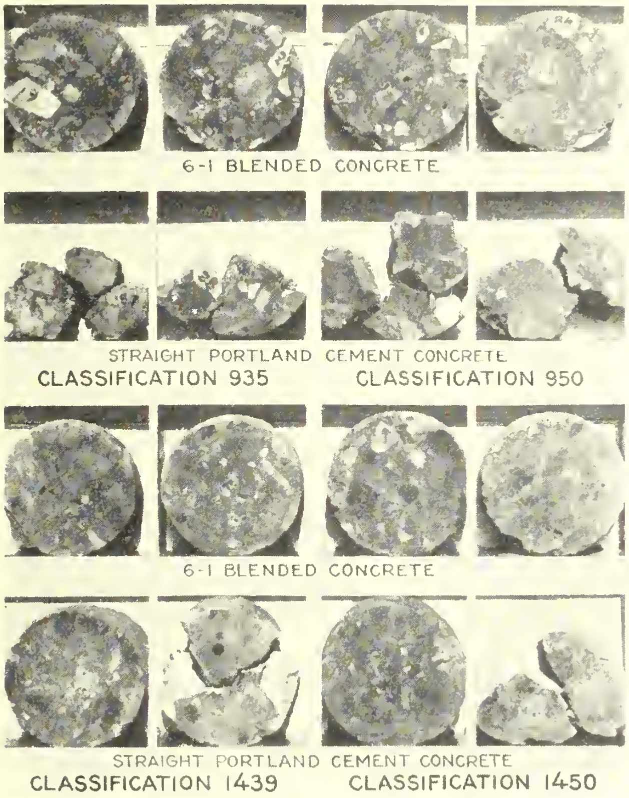

Figure 16.—Project A. Appearance of Blended and Straight Portland Cement Concrete Specimens after 50 Alternations of Freezing and Thawing in Water.

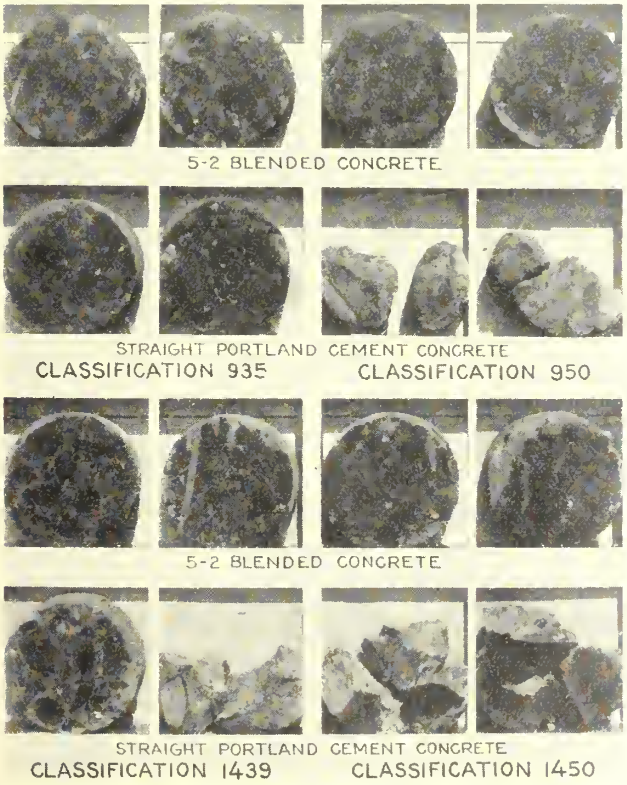

Figure 17.—Project B. Appearance of Blended and Straight Portland Cement Concrete Specimens after 50 Alternations of Freezing and Thawing in Water.

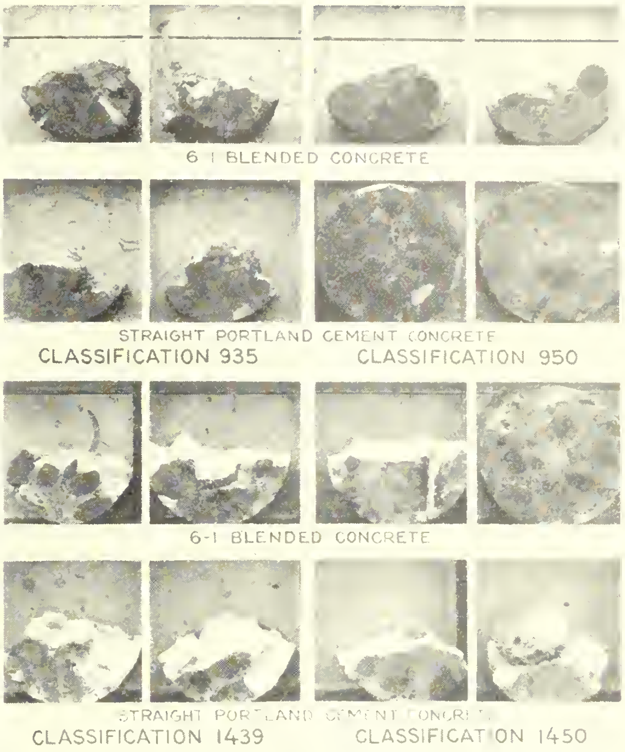

Figure 18.—Project C. Appearance of Blended and Straight Portland Cement Concrete Specimens after 50 Alternations of Freezing and Thawing in Water.

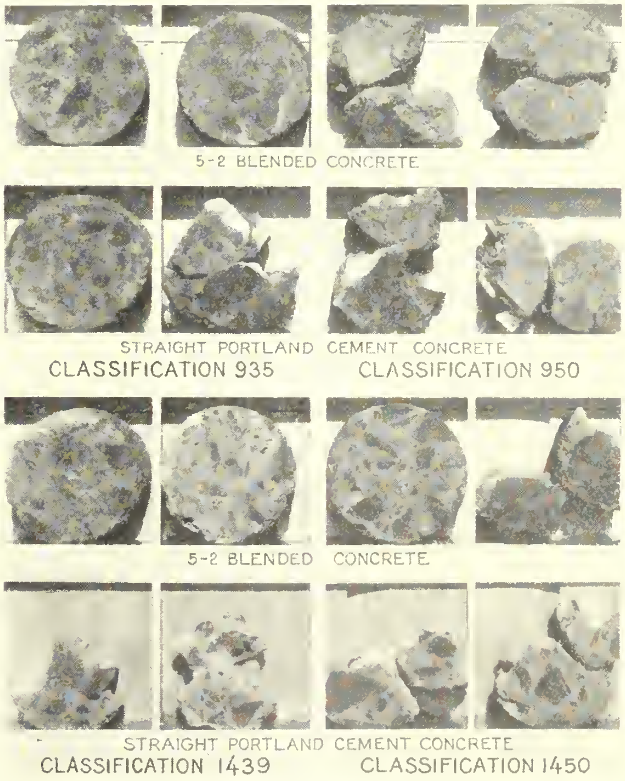

Figure 19.—Project D. Appearance of Blended and Straight Portland Cement Concrete Specimens after 50 Alternations of Freezing and Thawing in Water.

portland cement conforming to New York State specification classification 935, the two specimens in the top row containing the blended cement and the two specimens in the second row the straight portland cement. Similarly, the four specimens in the upper right quadrant contain cement conforming to the 950 classification, those in the lower left quadrant the 1439 cement, and those in the lower right quadrant the 1450 cement.

Referring to table 12 and to figures 12 to 15, inclusive, it will be noted that the use of the blend resulted in a marked increase in resistance to freezing in calcium chloride in Projects A and B, some improvement in Project D and no improvement in Project C. Projects A and B contained natural cement G in the blend, whereas natural cement H was used in Projects C and D.

A study of the individual results shows that in Project A all eight of the straight portland cement specimens disintegrated, while seven of the blended cement specimens remained sound. In Project B, six of the eight straight portland cement specimens disintegrated, whereas all eight of the blended specimens remained sound. In Project C five of the unblended specimens disintegrated as compared to six of the specimens containing the blended cement. In Project D all eight of the unblended specimens disintegrated, whereas three of the blended specimens disintegrated and one was unsound.

The comparative effect of freezing in calcium chloride and in water may be noted by comparing the ratings at 35 alternations as shown in the second and third columns of table 12. It will be observed that in Projects A and B the action in water was less severe than in calcium chloride. However, in Projects C and D the amount of disintegration was about the same for freezing in both liquids. The comparatively low resistance of Projects C and D to freezing in water would indicate that the quality of the concrete was inferior to that in Projects A and B.

Study of the comparative behavior of the various specimens when frozen in water for periods beyond 35 cycles may also be made by referring to table 12. In Projects A and B, virtually the same resistance is shown for the blended cements when frozen in water up to 150 cycles as was indicated when the specimens were frozen in calcium chloride for 35 cycles. However, it will be observed that the straight portland cement concrete in Project B was considerably more resistant to freezing and thawing in water at periods beyond 35 cycles than was the concrete in Project A. The concrete in Project C appeared to be definitely inferior, all specimens in both the straight portland and the blend having disintegrated at either 60 or 65 cycles. In Project D the behavior of the straight portland cement concrete specimens also indicated comparatively inferior concrete as compared to Projects A and B. However, the blended specimens from Project D were considerably more resistant than those containing straight portland cement although, as indicated above, the improvement was not as marked as in Projects A and B.

BLENDING FOUND TO INCREASE DURABILITY BY REDUCING BLEEDING

The comparative resistance of the individual specimens when frozen in water may be studied by referring to figure 20, which shows for each test disk the number of cycles at which the various stages of unsoundness were observed. Thus, for Project A, cement classification 1439, the figure shows that of the two unblended cement specimens, one developed questionable soundness at 50 alternations, became definitely unsound at 60, and disintegrated at 75. The other specimen was questionable at 20 alternations and disintegrated at 25. The two blended specimens were sound at 150 alternations. It will be observed that there is no consistent relation between composition of cement and resistance to freezing. This has already been commented upon and would appear to justify the method of rating the groups as a whole as shown in table 12.

An interesting feature of figure 20 is the close agreement which was observed in general between the behavior of the two specimens representing a given concrete. In many cases, failure progressed at almost an identical rate. The figure also shows the comparatively low resistance offered by the blended cement specimens from Projects C and D. Whether this was caused by the fact that natural cement H was used or by the fact that the concrete in these projects appeared, in general, to be of poorer quality than the concrete used in Projects A and B was not determined definitely. The fact remains, however, that the use of natural cement H did not improve the concrete in Project C at all and that while some improvement was shown for Project D, the results were not so positive as those indicated for Projects A and B, in both of which natural cement G was used.

Although there will be no extended discussion of the action of the blended cement concrete as compared to concrete containing only portland cement, there were certain physical differences noted in the concrete at the time the specimens were fabricated which will be discussed briefly. It has been observed in the field that there is less bleeding or water gain in concrete made with the blended cement than there is when portland cement is used alone. This observation led to a short laboratory investigation, the purpose of which was to determine the amount of water accumulating on the top of the fabricated specimens.

The concrete was proportioned in accordance with New York State highway specifications and contained approximately 6.3 sacks of cement per cubic yard. A slump of 2½ inches was used, requiring a water-cement ratio of 0.70.

Test specimens consisted of 6- by 6- by 21-inch beams molded in the usual way. After molding, the tops of the specimens were exposed to the laboratory air. Two specimens were made using 100 percent portland cement, two using a blend of 86 percent portland cement and 14 percent natural cement, and two using 72 percent portland cement and 28 percent natural cement.

At stated intervals after molding, the water which had accumulated at the top was carefully removed. The total water removed at the end of 2½ hours is shown in table 13, expressed as a percentage of the free water going into the concrete. This table also gives the results of a second test made several days later, showing the total water removed at the end of 4 hours.

From the results shown in the table it is evident that the substitution of natural cement for portland cement was effective in reducing bleeding. As will be noted, the bleeding was reduced as the amount of natural cement was increased, a replacement of 28 percent resulting in a reduction of more than one-half. Assuming that bleeding leaves the upper layer of concrete porous and weak as compared to the lower structure, it is apparent that the use of materials or methods of manipulation which will reduce bleeding should prove beneficial.

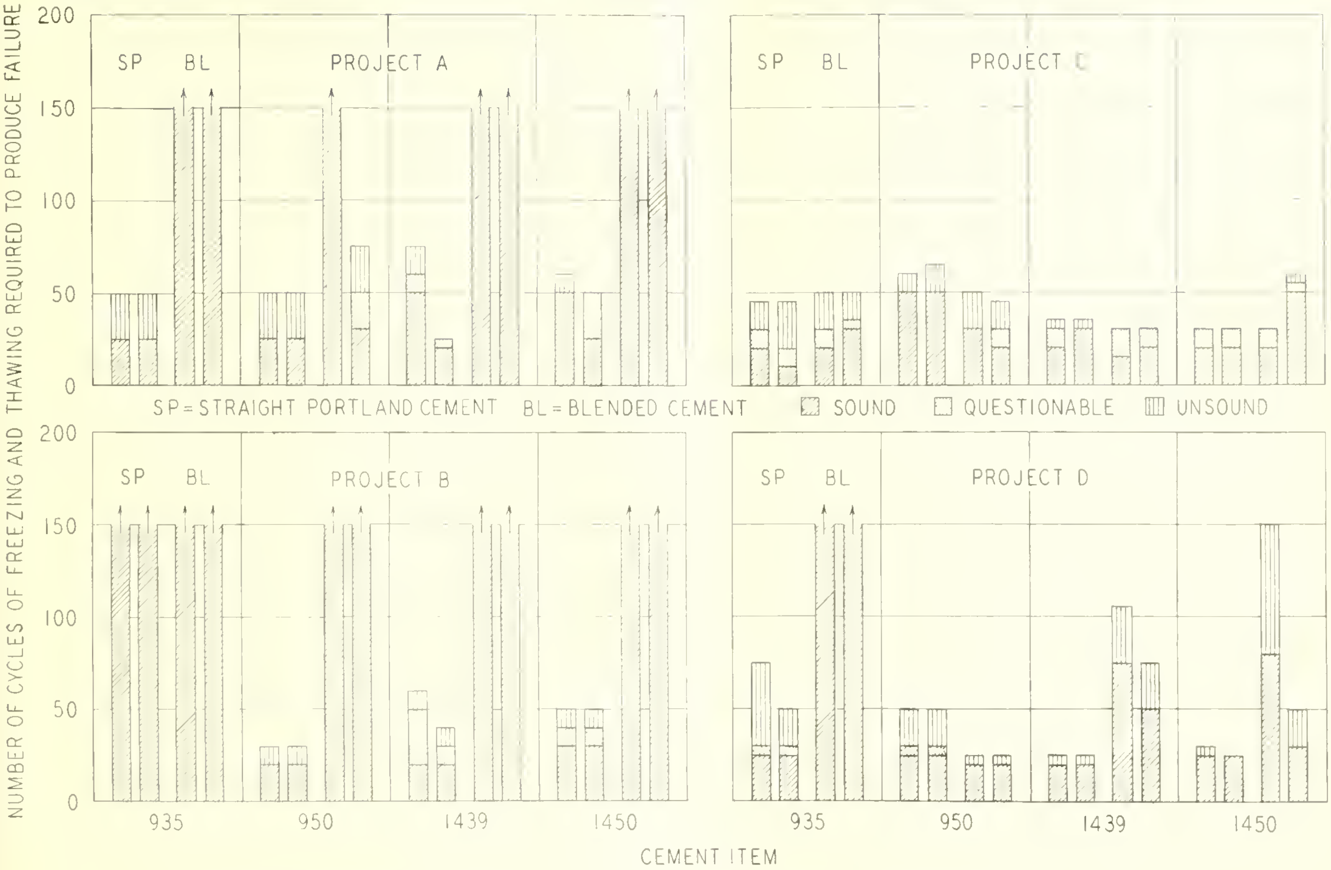

FIGURE 20.—EFFECT OF FREEZING AND THAWING CONCRETE CORE SPECIMENS IN WATER.

TABLE 13.—*Water removed from tops of 6 by 6 by 21-inch beam specimens in bleeding tests*

Cement blend		Water removed from specimens [1]	
Portland	Natural cement G	First test	Second test
	Percent	*Percent*	*Percent*
100 percent	0	7.6	7.4
86 percent	14	4.6	4.8
72 percent	28	3.5	2.9

[1] Based on total free water in concrete.

Data have been presented to show how the substitution of natural cement for portland cement materially increased the resistance to frost action of the top portions of the drilled cores.

In order to determine what effect water travel or bleeding has on durability, 2-inch disks were cut from the tops and also the bottoms of 9 identical cores and alternately frozen and thawed in a 10-percent calcium chloride solution until failure occurred. It was found that the average number of alternations required to produce failure was 20 for the tops of the cores and 35 for the bottoms, a difference of 15 alternations. All of the specimens were taken from Projects A and B and represented five different portland cements, none of which was blended with natural cement.

SUMMARY OF RESULTS

In the summary given below reference to a 1- or 2-sack replacement of portland cement by natural cement corresponds to a 14 or 28 percent replacement by weight, respectively. The values for strength are the averages of results of tests at all ages at which determinations were made.

A. The results of tests on specimens fabricated in the laboratory indicate that replacing a portion of the portland cement with natural cement:

1. Reduced the average crushing strength of 1:2 mortar specimens 7 percent for the 1-sack replacement and 19 percent for the 2-sack replacement (figs. 2 and 3).
2. Reduced the average flexural strength of 1:2 mortar specimens 6 percent for the 1-sack replacement and 15 percent for the 2-sack replacement (figs. 4 and 5).
3. Increased expansion of 1:2 mortar bars under continuous water storage up to 180 days (figs. 6 and 7).
4. Slightly increased contraction of 1:2 mortar bars under air storage conditions up to 120 days (figs. 6 and 7).
5. Considerably reduced the expansion of 1:2 mortar bars caused by repeated freezing and thawing, followed by heating, cooling and resaturation (figs. 8 and 9).
6. Decreased the bulk specific gravity and increased the water absorption of 1:2 mortar specimens (table 9).
7. Reduced the expansion of 1:5 mortar bars when subjected to the action of a 10-percent sulphate solution (fig. 10).
8. Increased the expansion of neat cement bars when subjected to the autoclave test at 420° F. for 3 hours (fig. 10).

9. Reduced the average crushing strength of concrete specimens containing 6 sacks of cement per cubic yard of concrete, 4 percent for the 1-sack replacement and 12 percent for the 2-sack replacement (fig. 11).

10. Increased the average flexural strength of concrete specimens containing 6 sacks of cement per cubic yard of concrete 1 percent for the 1-sack replacement and reduced the flexural strength 6 percent for the 2-sack replacement (fig. 11).

11. Decreased the average modulus of elasticity of concrete specimens containing 6 sacks of cement per cubic yard of concrete 1 percent for the 1-sack replacement and 5 percent for the 2-sack replacement (fig. 11).

B. The results of tests on cores taken from concrete pavements indicate that replacing a portion of the portland cement with natural cement considerably increased the resistance of the surface of the concrete to freezing and thawing in water and in a 10-percent solution of calcium chloride (table 12 and fig. 20).

In the tests covered by the above statement greater improvement in resistance was obtained by the use of natural cement G than by the use of natural cement H.

CONCLUSIONS

These tests indicate that, although the crushing and flexural strength of pavement concrete may be slightly reduced by the replacement of 14 or 28 percent of the portland cement with natural cement G, the resistance of the pavement surface to alternate freezing and thawing will be materially increased. No laboratory tests were made using natural cement H and tests of field specimens indicate that natural cement H did not improve the resistance of the concrete to the same extent as natural cement G.

HIGHWAY RESEARCH BOARD TO MEET SOON

The Eighteenth Annual Meeting of the Highway Research Board of the National Research Council will be held in Washington, D. C., Monday, November 28, to Friday, December 2, 1938. Reports on highway research investigations will be presented, and the formal meetings of the Board will be interspersed with open meetings for informal discussion of pertinent topics. A program of reports is to be announced by the Board in the near future.

DISPOSITION OF STATE MOTOR-FUEL TAX RECEIPTS, 1937

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments to undistributed funds etc. [1]	Tax on aviation gasoline deducted	Net total funds distributed [2]	Expenses of collection and administration	For other administrative purposes [3]	For State highway purposes						For local roads and streets [6]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes					
							Construction, maintenance, and administration [4]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads [4]	For work on city streets [7]	Service of local highway obligations	Total		To general funds [8]		For relief of unemployment or destitution	For education	For other specific purposes [9]	Total
									State highway bonds and notes	State assumed local obligations [5]	Total							Inspection fees, dealers licenses, etc.	Motor-fuel tax				
	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars	1,000 dollars
Alabama	13,350	61		13,411	82	56	4,195		2,392		2,392	6,587	6,686			6,686							
Arizona	4,333			4,333	96		2,829	104				2,933	*1,297			1,297						7	7
Arkansas	9,962			9,962	348	3	2,251		4,440	2,111	6,551	8,802	623		104	727		82					82
California	46,624	−1,205		45,419	168		26,373					26,373	*15,283	3,595		18,878							
Colorado	7,431	−30		7,401	89	23	4,528	236	570		570	5,334	1,955			1,955							
Connecticut	9,496	−209		9,287	51		7,406			1,830	1,830	9,236											
Delaware	2,034			2,034	11		[4] 1,364	133	41	329	370	1,867	(4)	13		13	138					5	5
Florida	22,466	16		22,482	24	395	9,123	39		3,300	3,300	12,462	103	38	6,293	6,434			2,542		604	21	3,167
Georgia	19,550			19,550	600		10,170			2,399	2,399	12,569	3,142			3,142					3,239		3,239
Idaho	4,037	43	6	4,074	15		4,056	3				4,059											
Illinois	36,266	42		36,308	353	261	11,836					11,836	*8,412	*7,970		16,382		168		4,409	2,899		7,476
Indiana	23,497	2,957		26,454	91	92	14,355					14,355	9,208	2,290		11,498		418					418
Iowa	13,047	55		13,102	81	23	3,921			3,269	3,269	7,190	*5,808			5,808							
Kansas	10,236	4		10,240	337	106	6,317	72	238	689	927	7,316	2,481			2,481							
Kentucky	12,671	46		12,717	48		12,461	153				12,614	55			55							
Louisiana	16,000	−3		15,997	87	74	1,100	87	7,521		7,521	8,708		262		262				4,622	1,122	1,122	6,866
Maine	5,550			5,550	19		3,364	165	1,443		1,443	4,972	559			559							
Maryland	9,857	13		9,870	34		3,929		[10] 1,456		1,456	5,385	930	2,786	735	4,451							
Massachusetts	19,836			19,836	50		5,145	247	[11] 2,292		2,292	7,684	2,933		[11] 345	3,278	[11] 833		6,600	[11] 1,391			7,991
Michigan	29,430	127	50	29,507	253		19,699		3,000		3,000	22,699	*6,550			6,550		5					5
Minnesota	15,493			15,493	49	100	10,051	144				10,195	*5,098			5,098		51					51
Mississippi	10,222	7		10,229	91		2,818	38	2,562		2,562	5,418	*4,267			4,267	453						
Missouri	11,217			11,217	55	51	6,913	205	3,919		3,919	11,037						74					74
Montana	4,581	68		4,649	19		4,064		566		566	4,630											
Nebraska	11,030		25	11,005	105		6,327					6,327	*3,020	331		3,351				1,222			1,222
Nevada	1,177			1,177	4		1,135	29		9	9	1,173											
New Hampshire	3,286	1		3,287	(12)		2,012	40	825		825	2,877	245		[13] 152	397	13						
New Jersey	21,593	193		21,786	148		1,614		7,633		7,633	9,247	3,443		557	4,000	68			6,322	1,238	763	8,323
New Mexico	4,022	−25		3,997	90	2	2,302		1,603		1,603	3,905											
New York [14]	61,915	−4,408		57,507	97		8,808	488	3,715		3,715	13,011	*8,545	1,680		10,225	2,081		32,093				32,093
North Carolina	23,393	−26		23,367	(15)	55	[4] 14,216	449	6,907	346	7,253	21,918	(4)					877	500			17	1,394
North Dakota	2,935	−133		2,802	12	61	1,621		189		189	1,810	905			905			14				14
Ohio	[16] 46,538	485		47,023	156		19,329					19,329	8,707	6,814		15,521					12,017		12,017
Oklahoma	13,772	−298		13,474	359		7,905					7,905	*3,352			3,352		4				1,854	1,858
Oregon	9,801	25	8	9,818	37		5,258	236	2,733		2,733	8,227	*1,467			1,467	33		54				54
Pennsylvania	55,720	[17] 8,100		63,820	298		37,889	1,246	2,973		2,973	42,108	*7,032	238		7,270	52			13,854		238	14,092
Rhode Island [14]	3,094	24		3,118	22		1,548	79	150		150	1,777	229			229	16		1,074				1,074
South Carolina	11,135	9	31	11,113	[18] 39	[18] 11	4,620		1,523	2,924	4,447	9,067	*1,812			1,812		184					184
South Dakota	4,155	343	3	4,495	49	46	2,661	3				2,664					36	38				1,662	1,700
Tennessee	18,938	−676	39	18,223	179	93	2,458		6,815	1,211	8,026	10,484	5,041			5,041		981				1,445	2,426
Texas	41,678	−52		41,626	642		20,492			10,246	10,246	30,738									10,246		10,246
Utah	3,424	69	36	3,457	12		3,312	133				3,445											
Vermont	2,323	183		2,506	3		1,291		310		310	1,601			897	897	5						
Virginia	16,122	37		16,159	(15)	47	[4] 15,318		518		518	15,836	[4] 267			267						9	9
Washington	15,282			15,282	26		5,951		[11] 119	52	171	6,122	*6,482	1,550	[11] 97	8,129				[11] 1,005			1,005
West Virginia	8,496			8,496	11		[4] 4,066		4,419		4,419	8,485	(4)										
Wisconsin	19,751	400		20,151	72	214	9,717			2,584	2,584	12,301	4,419	699		5,118	142		2,304				2,304
Wyoming	2,500		33	2,467	13		1,679	47	112		112	1,838	616			616							
District of Columbia	2,732			2,732	(19)									2,724		2,724		8					8
Total	761,998	6,243	231	768,010	5,425	1,713	359,797	4,376	70,984	31,299	102,283	466,456	130,972	30,990	9,180	171,142	3,870	2,890	45,181	32,825	31,365	7,143	119,404

[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See the following tables.

[3] Where reported separately from collection expenses, funds allotted for motor-fuel inspection, administration of motor vehicle department, and regulation of motor vehicles are shown in this column.

[4] The following allotments for construction and maintenance of county roads under State control are included in State highway purposes: Delaware, $259,000; North Carolina, $7,499,000; Virginia, $7,000,000; West Virginia, $1,231,000.

[5] Reimbursement to local units of Government for amounts spent on roads now on State system.

[6] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[7] This column shows specific allotments for city streets. Where reported separately, funds alloted for urban extensions of State highway system are included in allotments for State highway purposes.

[8] To State general funds except in Wisconsin, where amount went to towns, cities, and villages in lieu of personal property taxes formerly imposed on motor vehicles. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[9] For the following purposes: Arizona, irrigation engineering expenses; Delaware, beach protection; Florida, aviation; Louisiana, harbor improvement; New Jersey, service of institutional construction bonds, $520,000, and Department of Commerce and Aviation, $243,000; North Carolina, State probation commission; Oklahoma, service of general State debt; Pennsylvania, aircraft landing fields, $214,000, and cooperative work other departments, $24,000; South Dakota, payment on real estate bonds; Tennessee, debt service on nonhighway bonds; Virginia, aviation.

[10] Legislative restorations from proceeds of sale of general State bonds were applied to debt service payments thus reducing amounts necessary from current revenue.

[11] Includes debt service charges on emergency relief bond issues prorated in proportion to use of proceeds for State highway, local road, other highway, and nonhighway purposes.

[12] Paid out of motor-vehicle revenue, $3,000. See the following table.

[13] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[14] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts not otherwise dedicated.

[15] Included in cost of collecting motor-vehicle revenue. See the following table.

[16] Tax of $662,000 on non-motor-vehicle fuels not included.

[17] Amounts previously loaned to general fund for relief purposes entirely repaid during 1937.

[18] Estimated from fiscal-year appropriations.

[19] Paid out of general revenue. Amount not reported.

DISPOSITION OF STATE MOTOR-VEHICLE RECEIPTS, 1937

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration[3]	For other administrative purposes[4]	For State highway purposes						For local roads and streets[7]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes				
						Construction, maintenance, and administration[5]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads[5]	For work on city streets[8]	Service of local highway obligations	Total		To general funds[9]	For relief of unemployment or destitution	For education	For other specific purposes[10]	Total
								State highway bonds and notes	State-assumed local obligations[6]	Total											
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	4,439	135	4,574	484		2,085	406	835		835	3,326						764				764
Arizona	1,143		1,143	235		876	32				908										
Arkansas	3,241		3,241	101		770	130	1,518	722	2,240	3,140										
California	24,003	1,069	25,072	3,028	145	3,744	2,641	3,964		3,964	10,349	*3,794			3,794		7,756				7,756
Colorado	2,603	5	2,608	466	55	898	49	119		119	1,066	1,021			1,021						
Connecticut	6,691	−407	6,284	1,124		1,501	325		371	371	2,197	2,963			2,963						
Delaware	1,202	75	1,277	104		[5] 790	77	24	191	215	1,082	(5)	8		8	80				3	3
Florida	6,196		6,196	464	224														5,508		5,508
Georgia	2,368		2,368	264		941	320		222	222	1,483	621			621						
Idaho	2,498	−25	2,473	71		289	101				390	*2,012			2,012						
Illinois	21,430	422	21,852	831	361	8,907	1,436	8,837	382	9,219	19,562	891			891		207				207
Indiana	9,827	29	9,856	931		3,907	446				4,353	1,561	390		1,951		2,621				2,621
Iowa	11,918	67	11,985	1,387		5,780			4,818	4,818	10,598										
Kansas	4,537	−260	4,277	709		2,301	26	86	251	337	2,664	904			904						
Kentucky	5,125	−4	5,121	469	26	2,174	27				2,201	600			600		1,825				1,825
Louisiana	4,710	−201	4,509	73		2,946	325	447		447	3,718		718		718						
Maine	3,865	−1	3,864	151		2,259	110	969		969	3,338	375			375						
Maryland	5,577	−1,006	4,571	522	56	1,694	378	[11] 1,107		1,107	3,179		814		814						
Massachusetts	6,875	213	7,088	1,580	35	2,136	103	[12] 951		951	3,190	1,217		[12] 143	1,360	[12] 346		[12] 577			577
Michigan	22,085	−209	21,876	1,700		455	321				776	*19,129			19,129		271				271
Minnesota	8,867	35	8,902	428		4,214	137	1,748	2,261	4,009	8,360						114				114
Mississippi	2,248	−11	2,237	108		212					212	*1,917			1,917						
Missouri	9,638	−10	9,628	615		5,645	168	3,200		3,200	9,013										
Montana	1,532	95	1,627	73			159				159	1,354	41		1,395						
Nebraska	2,603	−159	2,444	201	21	657	75				732	1,482			1,482		8				8
Nevada	282		282	25		166	4	87		87	257										
New Hampshire	2,347	56	2,403	126	3	1,770	139	4		4	1,913	216		[13] 133	349	12					
New Jersey	19,271	−23	19,248	1,767		3,897					3,897	5,794		1,345	7,139	164		6,281			6,281
New Mexico	1,544	176	1,720	93	114	599	114				713	218			218		582				582
New York [14]	52,901	−288	52,613	2,736	176	15,279	847	6,445		6,445	22,571	*10,692	4,949		15,641	3,609	7,880				7,880
North Carolina	8,855	−637	8,218	446		[5] 5,041	159	2,449	123	2,572	7,772	(5)									
North Dakota	1,581	−19	1,562	130	22	632	121	62		62	815	595			595						
Ohio	25,635	1,284	26,919	1,403		6,273	501				6,774	*13,836	4,532		18,368					374	374
Oklahoma	5,584	8	5,592	930	180	1,603	294				1,897	*2,585			2,585						
Oregon	3,378	19	3,397	460		1,579	71	821		821	2,471	*440			440	10	16				16
Pennsylvania	38,332	[15] 10,788	49,120	1,990		41,882	1,377	3,287		3,287	46,546		262		262	58				264	264
Rhode Island [14]	2,790	69	2,859	258		1,300	67	126		126	1,493	193			193	13	902				902
South Carolina	1,690		1,690	142		645	282	213	408	621	1,548										
South Dakota	1,650	−2	1,648	77	4	322					322	1,242			1,242	3					
Tennessee	4,233	8	4,241	261		3,537	443				3,980										
Texas	19,684	25	19,709	993	320	6,282	443				6,725	*11,671			11,671						
Utah	1,049	−241	808	123	12		11	592		592	603						70				70
Vermont	2,410	175	2,585	87		1,239	94	298		298	1,631			862	862	5					
Virginia	6,153	−2	6,151	595		4,976	205	375		375	5,556										
Washington	4,402	52	4,454	347		2,604	1,490				4,094	13			13						
West Virginia	6,162		6,162	216		[5] 2,845	10	3,091		3,091	5,946	(5)									
Wisconsin	12,984	−506	12,478	774	32	5,707			1,518	1,518	7,225	2,596	410		3,006	83	1,358				1,358
Wyoming	597	−4	593	4		411	12	166		166	589										
District of Columbia	878	−2	876	86	75												715				715
Total	399,613	10,788	410,401	30,188	1,861	163,770	14,476	41,821	11,267	53,088	231,334	89,932	12,124	2,483	104,539	4,383	25,089	6,858	5,508	641	38,096

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[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See tables that precede and follow this table.

[3] Collection expenses in many States include service charges deducted by county and local collectors.

[4] Where reported separately from collection expenses, funds allotted for collection of motor-fuel tax, payments to auto theft fund, and miscellaneous expenses of motor-vehicle regulation are shown in this column.

[5] The following allotments for construction and maintenance of county roads under State control are included in State highway purposes: Delaware, $150,000; North Carolina, $2,659,000; West Virginia, $863,000.

[6] Reimbursement to local units of government for amounts spent on roads now on State system.

[7] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[8] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[9] To State general funds except in the following States: Alabama, county and municipal general funds; California, general funds of counties and cities, $3,821,000; New Mexico, county general funds, $364,000; Wisconsin, towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles, $1,353,000. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[10] For the following purposes: Delaware, beach protection; Ohio, hospitalization of indigent persons injured in motor-vehicle accidents; Pennsylvania, aircraft landing fields, $237,000, and cooperative work other departments, $27,000.

[11] Legislative restorations from proceeds of sale of general State bonds were applied to debt service payments, thus reducing amounts necessary from current revenue.

[12] Includes debt service charges on emergency relief bond issues prorated in proportion to use of proceeds for State highway, local road, other highway, and nonhighway purposes.

[13] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[14] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts not otherwise dedicated.

[15] Amounts previously loaned to general fund for relief purposes entirely repaid during 1937

DISPOSITION OF STATE MOTOR-CARRIER TAX RECEIPTS, 1937

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year	Adjustments due to undistributed funds, etc.[1]	Net total funds distributed[2]	Expenses of collection and administration	For State highway purposes						For local roads and streets[5]				For other highway purposes (park and forest roads, etc.)	For nonhighway purposes			
					Construction, maintenance, and administration[3]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads[3]	For work on city streets[6]	Service of local highway obligations	Total		To general funds	For relief of unemployment or destitution	For education	Total
							State highway bonds and notes	State-assumed local obligations[4]	Total										
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	216	1	217	58	159					159									
Arizona	162	−2	160	13	142	5				147									
Arkansas	2		2		2					2									
California	3,048	−41	3,007	544	46					46	16			16		2,401			2,401
Colorado	621	21	612	98	238	13	31		31	282	262			262					
Connecticut	211	−15	196		88			22	22	110	86			86					
Delaware	(7)																		
Florida	262	−4	258	32							10		210	220				6	6
Georgia	155	226	381	111	218			52	52	270									
Idaho	106	37	143	30	81	32				113									
Illinois	(7)																		
Indiana	682	−148	534	134	400					400									
Iowa	537		537	135							*402			402					
Kansas	1,163		1,163	281	549	38	20	60	80	667	215			215					
Kentucky	394	85	479	57	415	5				420	2			2					
Louisiana	3		3	3															
Maine	22	−4	18	18															
Maryland	(8)																		
Massachusetts	55		55	55															
Michigan	403	−58	345	197	148					148									
Minnesota	23		23	23															
Mississippi	158		158	4							*154			154					
Missouri	521	411	932	79	497	15	281		281	793	60			60					
Montana	46	−11	35	35															
Nebraska	27		27	27															
Nevada	198		198	12	181	5				186									
New Hampshire	4		4	4															
New Jersey	91	−7	84		22					22	19		7	26	1		35		35
New Mexico	153	−9	144	34	104	6				110									
New York	(7)																		
North Carolina	251	−18	233	1	[3] 151	4	73	4	77	232	(3)								
North Dakota	23	10	33	23	10					10									
Ohio	492	214	706	108	571					571	27			27					
Oklahoma	1,271	−5	1,266	91	843					843	332			332					
Oregon	1,094	−36	1,058	211	443	43	230		230	716	*124			124	2	5			5
Pennsylvania	8		8		8					8									
Rhode Island	9		9	9															
South Carolina	189	−54	135	32	93					93		10		10					
South Dakota	484	−15	469	41	423					423					5				
Tennessee	369	2	371	69	234					234						68			68
Texas	109		109	102	7					7									
Utah	113	−15	98	11	81					81						6			6
Vermont	(7)																		
Virginia	200	−3	197	26	135		10		10	145						26			26
Washington	245		245	245															
West Virginia	91		91		36		55		55	91									
Wisconsin	1,559	−56	1,503	356												1,147			1,147
Wyoming	227	−1	226	33	188	5				193									
District of Columbia	219		219													219			219
Total	16,216	505	16,721	3,342	6,513	171	700	138	838	7,522	1,709	10	217	1,936	8	3,872	35	6	3,913

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[1] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[2] In many States the proceeds of highway user taxes are placed in a common fund from which a distribution is made. The amounts so distributed have been prorated in proportion to the receipts not otherwise dedicated. See 2 preceding tables.

[3] Approximately $80,000 allotted for use on county roads under State control in North Carolina included in State highway purposes.

[4] Reimbursement to local units of Government for amounts spent on roads now on State system.

[5] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[6] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

[7] No special taxes on motor carriers reported.

[8] Ton-mile and passenger-mile taxes paid by motor carriers in lieu of registration fees included in motor-vehicle receipts, preceding table.

DISPOSITION OF RECEIPTS FROM STATE IMPOSTS ON HIGHWAY USERS, 1937

[Compiled for calendar year from reports of State authorities]

State	Net total receipts of calendar year [1]	Adjustments due to undistributed funds etc.[2]	Tax on aviation gasoline deducted	Net total funds distributed	Expenses of collection and administration[3]	For State highway purposes						For local roads and streets [6]				For nonhighway purposes						
						Construction, maintenance, and administration [4]	State highway police	Service of State highway obligations			Total for State highway purposes	For work on county and local roads [4]	For work on city streets [7]	Service of local highway obligations	Total	For other highway purposes (park and forest roads, etc.)	To general funds [8]		For relief of unemployment or destitution	For education	For other specific purposes [9]	Total
								State highway bonds and notes	State-assumed local obligations [5]	Total							Motor-fuel inspection fees, dealers' licenses, etc.	All other highway user imposts				
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
Alabama	18,005	197		18,202	680	6,439	406	3,227		3,227	10,072	6,686			6,686			764				764
Arizona	5,638	−2		5,636	344	3,847	141				3,988	*1,297			1,297						7	7
Arkansas	13,205			13,205	452	3,023	130	5,958	2,833	8,791	11,944	623		104	727		82					82
California	73,675	−177		73,498	3,885	30,163	2,641	3,964		3,964	36,768	*19,093	3,595		22,688			10,157				10,157
Colorado	10,655	−4		10,651	731	5,664	298	720		720	6,682	3,238			3,238							
Connecticut	16,398	−631		15,767	1,175	8,995	325		2,223	2,223	11,543	3,049			3,049							
Delaware	3,236	75		3,311	115	[4] 2,154	210	65	520	585	2,949	([4])	21		21	218					8	8
Florida	28,924	12		28,936	1,139	9,123	39		3,300	3,300	12,462	113	38	6,503	6,654			2,542		6,118	21	8,681
Georgia	22,073	226		22,299	975	11.329	320		2,673	2,673	14,322	3,763			3,763					3,239		3,239
Idaho	6,641	55	6	6,690	116	4,426	136				4,562	*2,012			2,012							
Illinois	57,696	464		58,160	1,806	20,743	1,436	8,837	382	9,219	31,398	*9,303	*7,970		17,273		168	207	4,409	2,899		7,683
Indiana	34,006	2,838		36,844	1,248	18,662	446				19,108	10.769	2,680		13,449		418	2,621				3,039
Iowa	25,502	122		25,624	1,626	9,701			8,087	8,087	17,788	*6,210			6,210							
Kansas	15,936	−256		15,680	1,433	9,167	136	344	1,000	1,344	10,647	3,600			3,600							
Kentucky	18,190	127		18,317	600	15,050	185				15,235	657			657			1,825				1,825
Louisiana	20,713	−204		20,509	237	4,046	412	7,968		7,968	12,426		980		980				4,622	1,122	1,122	6,866
Maine	9,437	−5		9,432	188	5,623	275	2,412		2,412	8,310	934			934							
Maryland	15.434	−993		14,441	612	5,623	378	[10] 2,563		2,563	8,564	930	3,600	735	5,265							
Massachusetts	26,766	213		26,979	1,720	7,281	350	[11] 3,243		3,243	10,874	4,150		[11] 488	4,638	[11] 1,179		6,600	[11] 1,968			8,568
Michigan	51,918	−140	50	51,728	2,150	20,302	321	3,000		3,000	23,623	*25,679			25,679		5	271				276
Minnesota	24,383	35		24,418	600	14,265	281	1,748	2,261	4,009	18,555	*5,098			5,098		51	114				165
Mississippi	12,628	−4		12,624	203	3,030	38	2,562		2,562	5,630	*6,338			6,338	453						
Missouri	21,376	401		21,777	800	13,055	388	7,400		7,400	20,843	60			60		74					74
Montana	6,159	152		6,311	127	4,064	159	566		566	4,789	1,354	41		1,395							
Nebraska	13,660	−159	25	13,476	354	6,984	75				7,059	*4,502	331		4,833			8	1,222			1,230
Nevada	1,657			1,657	41	1,482	38	87	9	96	1,616											
New Hampshire	5,637	57		5,694	133	3,782	179	829		829	4,790	461		[12] 285	746	25						
New Jersey	40,955	163		41,118	1,915	5,533		7,633		7,633	13,166	9,256		1,909	11,165	233			12,638	1,238	763	14,639
New Mexico	5,719	142		5,861	333	3,005	120	1,603		1,603	4,728	218			218			582				582
New York [13]	114,816	−4,696		110,120	3,009	24,087	1,335	10,160		10,160	35,582	*19,237	6,629		25,866	5,690		39,973				39,973
North Carolina	32,499	−681		31,818	502	[4] 19,408	612	9,429	473	9,902	29,922	([4])					877	500			17	1,394
North Dakota	4,539	−142		4,397	248	2,263	121	251		251	2,635	1,500			1,500			14				14
Ohio	72,665	1,983		74,648	1,667	26,173	501				26,674	*22,570	11,346		33,916					12,017	374	12,391
Oklahoma	20,627	−295		20,332	1,560	10,351	294				10,645	*6,269			6,269		4				1,854	1,858
Oregon	14,273	8	8	14,273	708	7,280	350	3,784		3,784	11,414	*2,031			2,031	45		75				75
Pennsylvania	94,060	[14]18,888		112,948	2,288	79,779	2,623	6,260		6,260	88,662	*7,032	500		7,532	110			13,854		502	14,356
Rhode Island [13]	5,893	93		5,986	289	2,848	146	276		276	3,270	422			422	29		1,976				1,976
South Carolina	13,014	−45	31	12,938	224	5,358	282	1,736	3,332	5,068	10,708	*1,812	10		1,822		184					184
South Dakota	6,289	326	3	6,612	217	3,406	3				3,409	1,242			1,242	44	38				1,662	1,700
Tennessee	23,540	−666	39	22,835	602	6,229	443	6,815	1,211	8,026	14,698	5,041			5,041		981	68			1,445	2,494
Texas	61,471	−27		61,444	2,057	26,781	443		10,246	10,246	37,470	*11,671			11,671					10,246		10,246
Utah	4,586	−187	36	4,363	158	3,393	144	592		592	4,129							76				76
Vermont	4,733	358		5,091	90	2,530	94	608		608	3,232			1,759	1,759	10						
Virginia	22,475	32		22,507	668	[4] 20,429	205	903		903	21,537	[4] 267			267			26			9	35
Washington	19,929	52		19,981	618	8,555	1,490	[11] 119	52	171	10,216	*6,495	1,550	[11] 97	8,142				[11] 1,005			1,005
West Virginia	14,749			14,749	227	[4] 6,947	10	7,565		7,565	14,522	([4])										
Wisconsin	34,294	−162		34,132	1,448	15,424			4,102	4,102	19,526	7,015	1,109		8,124	225		4,809				4,809
Wyoming	3,324	−5	33	3,286	50	2,278	64	278		278	2,620	616			616							
District of Columbia	3,829	−2		3,827	161								2,724		2,724		8	934				942
Total	1,177,827	17,536	231	1,195,132	42,529	530,080	19,023	113,505	42.704	156,209	705,312	222.613	43,124	11,880	277,617	8,261	2,890	74,142	39,718	36,879	7,784	161,413

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[1] Includes receipts from motor-fuel taxes, motor-vehicle fees and fines, and special imposts on motor vehicles operated for hire (motor-carrier taxes). See preceding tables, which give distribution of receipts separately.

[2] Amounts distributed during the calendar year often differ from actual collections because of undistributed funds and lag between accounts of collecting and expending agencies.

[3] Includes expenses of collection and administration of motor-fuel tax, motor-vehicle fees, and motor-carrier taxes, and miscellaneous expenses of motor-vehicle regulation.

[4] The following allotments for construction and maintenance of county roads under State control are included in State highway purposes: Delaware, $409,000; North Carolina, $10,238,000; Virginia, $7,000,000; West Virginia, $2,094,000.

[5] Reimbursement to local units of government for amounts spent on roads now on State system.

[6] In States indicated by star (*) law provides that these funds may also be used for service of local highway obligations. Amounts so used not reported separately. In Colorado funds may be used on both State and local roads.

[7] This column shows specific allotments for city streets. Where reported separately, funds allotted for urban extensions of State highway system are included in allotments for State highway purposes.

consin, towns, cities, and villages in lieu of personal property tax formerly imposed on motor vehicles, $3,657,000. Allocations to local general funds may have been used in part for highways, but such amounts not reported.

[9] For the following purposes: Arizona, irrigation engineering expenses; Delaware, beach protection; Florida, aviation; Louisiana, harbor improvement; New Jersey, service of institutional construction bonds, $520,000, and Department of Commerce and Navigation, $243,000; North Carolina, State probation commission; Ohio, hospitalization of indigent persons injured in motor-vehicle accidents; Oklahoma, service of general State debt; Pennsylvania, aircraft landing fields, $451,000, and cooperative work other departments, $51,000; South Dakota, payments on real estate bonds; Tennessee, debt service on nonhighway bonds; Virginia, aviation.

[10] Legislative restorations from proceeds of sale of general State bonds were applied to debt service payments, thus reducing amounts necessary from current revenue.

[11] Includes debt service charges on emergency relief bond issues prorated in proportion to use of proceeds for State highway, local road, other highway, and nonhighway purposes.

[12] Service of highway relief bonds, a State obligation incurred for improvement of local roads.

[13] Appropriations for highway purposes out of State general fund have been credited against payments of motor-fuel tax and motor-vehicle fees to the general fund and prorated in proportion to net receipts not otherwise

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF SEPTEMBER 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,659,461	$ 735,930	17.6	$ 8,126,378	$ 4,053,960	344.6	$ 2,176,494	$ 1,085,955	94.4	$ 2,875,596
Arizona	988,010	787,011	69.3	1,289,778	941,811	44.2	194,821	120,880	4.3	1,451,953
Arkansas	541,415	539,727	63.1	1,733,621	1,730,820	111.3	1,632,270	1,630,454	97.6	1,063,230
California	2,950,086	1,547,565	73.9	10,179,207	5,360,556	181.0	2,137,333	1,140,456	49.6	1,808,453
Colorado	1,466,214	806,752	64.1	2,917,207	1,605,878	73.3	920,810	511,980	31.5	1,794,558
Connecticut				1,350,040	660,511	14.7	244,930	120,990	2.0	1,536,794
Delaware	75,320	37,660		1,010,202	500,965	23.5	231,920	115,910	.2	1,184,277
Florida	480,110	240,055	15.1	3,787,138	1,893,569	79.7	468,000	234,000	.3	2,981,176
Georgia	2,807,740	1,398,558	147.4	5,981,044	2,990,522	277.9	768,970	384,485	45.1	5,527,478
Idaho	1,299,877	775,218	116.6	1,830,239	1,088,878	111.4				1,134,504
Illinois	1,782,500	908,384	48.7	11,668,268	5,832,370	262.2	3,149,550	1,574,774	91.8	2,328,326
Indiana	2,035,190	1,017,595	55.5	4,891,291	2,445,645	109.0	1,793,604	896,672	36.1	2,350,710
Iowa	2,649,293	1,232,519	81.0	7,195,830	3,242,183	237.9	1,763,024	827,300	57.2	349,747
Kansas	2,117,710	1,058,754	104.9	5,045,834	2,479,217	678.3	3,602,952	1,800,473	191.5	2,955,696
Kentucky	2,229,306	1,101,911	88.4	5,150,474	2,575,237	146.5	1,520,780	760,390	45.4	2,073,578
Louisiana	496,596	248,267	15.8	12,060,405	2,403,818	52.8	1,648,572	721,592	28.6	2,347,042
Maine	1,911,860	955,864	45.6	1,792,486	891,888	34.7	435,005	217,501	10.3	230,174
Maryland	308,000	154,000	5.3	1,891,567	944,021	29.0	1,688,981	820,830	28.3	1,670,916
Massachusetts	567,951	283,974	3.8	2,644,499	1,322,246	13.5	855,151	425,990	8.9	2,316,555
Michigan	2,790,200	1,395,100	86.6	6,481,208	3,168,694	124.2	1,736,469	863,322	40.5	1,713,365
Minnesota	567,280	256,994	10.5	7,088,576	3,528,072	345.5	2,123,524	1,053,619	103.4	1,924,238
Mississippi	759,208	352,662	31.9	8,326,480	3,205,471	368.9	1,409,600	351,970	71.1	3,115,942
Missouri	2,018,904	1,004,195	65.1	3,985,170	1,950,258	101.3	4,096,238	1,778,114	159.4	3,370,342
Montana	1,383,975	778,153	69.4	674,888	379,264	19.4	317,839	178,783	11.0	4,363,986
Nebraska	2,097,633	1,048,733	248.7	6,178,086	3,079,442	457.8	2,070,308	401,601	52.3	2,553,158
Nevada	1,080,635	936,162	106.1	623,465	534,727	75.2	468,897	406,166	8.2	1,217,185
New Hampshire	711,007	351,969	19.8	558,750	278,256	4.9	143,230	71,614	1.5	1,138,227
New Jersey				3,028,331	1,511,228	21.7	305,990	152,370	3.3	2,537,873
New Mexico	1,157,176	705,751	145.4	1,884,391	1,249,455	119.5	210,862	128,604	6.6	798,787
New York	4,890,956	2,398,750	105.1	14,251,023	7,037,212	238.6	2,024,030	982,665	25.0	2,817,777
North Carolina	2,642,358	1,304,224	136.1	6,837,996	3,304,827	328.2	1,769,590	873,470	95.0	1,933,713
North Dakota	509,340	460,000	50.2	2,889,891	2,643,078	212.0	136,652	73,189	10.2	3,567,977
Ohio	2,098,470	1,047,200	28.9	8,541,422	4,242,300	89.8	5,579,690	2,772,519	49.6	5,846,598
Oklahoma	2,390,057	1,259,286	81.8	5,266,416	2,748,303	185.5	1,401,390	744,465	58.6	2,656,311
Oregon	1,240,688	707,212	49.2	1,747,565	1,058,367	47.5	425,782	259,860	56.7	1,982,244
Pennsylvania	4,465,212	2,230,026	74.9	5,563,978	2,759,958	78.3	6,560,257	3,261,179	60.0	2,672,558
Rhode Island	202,800	101,400	1.7	939,242	469,621	14.3	287,710	143,855	2.1	1,028,924
South Carolina	2,730,437	1,209,193	152.5	3,370,242	1,503,432	125.8	2,003,011	900,100	56.7	1,165,475
South Dakota	611,862	353,755	73.9	4,089,492	2,261,560	384.5	1,580,520	872,770	117.3	2,720,117
Tennessee	1,623,080	811,540	56.0	4,868,714	2,434,357	128.5	868,860	433,930	15.4	4,310,972
Texas	6,823,031	3,387,715	440.9	10,183,495	5,046,449	509.9	4,263,605	1,886,002	212.5	6,275,668
Utah	725,860	521,130	102.4	1,613,295	1,146,535	55.2	354,730	251,750	10.0	894,864
Vermont	819,759	364,476	23.3	971,790	461,233	25.7	328,570	164,095	5.3	148,696
Virginia	2,594,411	1,296,500	86.2	4,646,135	2,320,402	157.1	1,510,354	752,827	40.0	688,815
Washington	2,249,597	1,183,100	46.2	3,138,123	1,644,159	45.8	804,454	423,000	13.5	828,161
West Virginia	703,299	518,649	26.0	1,910,644	1,203,321	57.3	708,719	342,923	9.8	2,176,030
Wisconsin	2,439,158	1,181,453	72.0	6,356,365	3,187,350	206.9	987,244	479,920	28.8	1,551,246
Wyoming	1,163,657	715,911	140.0	1,377,252	845,491	154.6	238,950	147,620	24.8	670,022
District of Columbia										
Hawaii	369,705	184,853	6.6	1,270,235	626,047	20.4	28,450	11,865	.3	1,202,411
Puerto Rico				1,128,739	562,390	20.5	28,590	14,245	1.9	657,740
TOTALS	80,226,394	41,895,836	3,553.5	220,836,907	109,355,354	7,550.3	70,007,282	34,569,044	2,173.9	106,510,185

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF SEPTEMBER 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama				$ 562,000	$ 280,700	37.8	$ 82,605	$ 40,200	6.2	$ 732,072
Arizona	$ 281,361	$ 187,459	18.6	45,324	27,889	2.0	76,142	45,177	4.7	395,957
Arkansas				19,426	12,863		107,411	107,356	12.6	743,889
California	392,268	226,045	34.8	1,228,447	670,195	62.6	421,417	230,347	33.8	720,639
Colorado	425,450	236,550	31.6	417,036	230,283	19.5	88,360	32,000	4.8	422,735
Connecticut				57,470	28,735	1.2	29,410	14,510	.3	275,033
Delaware							103,704	51,852	16.9	195,023
Florida				20,122	10,061		105,900	52,950	.7	611,841
Georgia	129,837	63,897	17.6	498,260	249,130	58.4	256,080	128,040	37.1	817,655
Idaho	243,115	110,501	39.8	229,211	97,855	14.7	35,697	17,955	2.3	247,261
Illinois	553,000	276,500	44.7	1,672,992	782,496	125.2	648,400	324,200	43.7	643,719
Indiana	56,600	24,600	9.2	772,700	334,350	76.9	1,059,148	516,702	97.1	366,786
Iowa										1,298,449
Kansas	43,670	21,835	.5	67,822	33,911	7.6	268,154	134,076	48.4	1,129,149
Kentucky	309,086	104,551	52.0	630,278	186,311	50.4	1,262,037	335,098	94.9	188,466
Louisiana				155,231	72,985	11.6	878,653	371,430	75.4	279,721
Maine	174,400	87,200	11.0	327,126	163,563	19.7	138,600	63,433	6.0	14,377
Maryland				6,264	3,132		84,800	29,605	8.7	379,739
Massachusetts				5,300	2,650		143,050	71,315	2.4	572,435
Michigan				616,866	308,433	46.1	305,290	152,645	25.8	1,074,461
Minnesota	130,131	61,694	21.3	321,169	151,900	24.9	318,140	158,740	40.0	1,007,246
Mississippi				104,800	52,400	4.0	194,200	97,100	19.8	739,427
Missouri	224,357	111,427	34.3	231,370	115,290	15.3	616,650	225,705	84.1	612,827
Montana				13,983	7,865					1,027,170
Nebraska	149,082	74,541	21.6	451,070	225,535	72.9	247,552	120,556	47.4	575,420
Nevada	228,041	194,840	30.8	119,166	100,152	32.7	94,427	81,932	22.1	117,369
New Hampshire	97,426	48,277	1.7	111,335	55,301	2.8	39,820	19,880	1.5	123,417
New Jersey				123,040	61,520	2.4	223,920	103,630	2.7	507,623
New Mexico	289,750	176,717	14.4	327,188	190,070	22.5	299,156	162,318	15.2	277,667
New York	1,484,731	739,812	102.8	1,420,300	692,550	94.6	1,101,600	469,450	42.2	458,403
North Carolina	54,380	27,190	10.1	1,004,664	502,310	97.0	293,440	136,520	19.6	415,835
North Dakota				106,900	57,253	26.8	91,100	48,792	16.2	681,207
Ohio				184,400	92,200	3.8	60,600	29,750	1.8	1,709,691
Oklahoma	12,200	6,491		396,738	209,979	34.2	449,030	215,734	29.9	752,711
Oregon	279,315	169,410	44.7	152,117	92,122	9.3	60,504	28,000	6.6	463,622
Pennsylvania	758,719	369,060	54.8	1,201,117	581,021	73.0	1,347,074	654,837	72.8	510,970
Rhode Island	66,840	33,420	3.5	68,848	34,424	2.6	187,830	93,915	3.1	26,132
South Carolina	50,120	22,700	6.6	752,519	319,662	84.9	482,027	191,400	46.6	144,918
South Dakota				11,300	6,250					816,436
Tennessee	53,200	26,600	1.3	400,886	184,973	23.4	275,920	101,810	14.6	746,107
Texas	690,705	313,110	122.6	2,070,709	919,140	220.8	734,944	306,775	111.0	1,585,744
Utah	226,240	125,370	28.1	372,398	177,140	24.8	170,330	75,400	26.9	172,460
Vermont	163,580	71,150	9.7	101,376	49,153	4.8	114,700	56,200	3.8	33,093
Virginia	255,430	125,500	28.8	728,647	315,041	53.1	243,230	113,931	20.6	337,527
Washington	266,218	139,900	19.9	310,853	163,278	42.2	395,197	208,000	24.6	219,378
West Virginia	28,900	14,450	1.4	215,150	107,575	20.0	122,300	61,150	6.2	366,499
Wisconsin	46,947	23,405	1.4	924,481	441,190	38.5	252,117	115,380	14.4	632,253
Wyoming	290,340	179,400	33.7	333,680	206,180	20.9	124,440	76,890	14.0	122,723
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico				244,000	121,950	13.7	35,610	17,455	1.5	107,470
TOTALS	8,455,439	4,393,602	853.3	20,192,329	9,757,091	1,602.0	14,670,716	6,690,141	1,231.0	26,619,502

U. S. GOVERNMENT PRINTING OFFICE 1938

PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 10 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T . . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . . . Bibliography on Highway Safety.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF SEPTEMBER 30, 1938

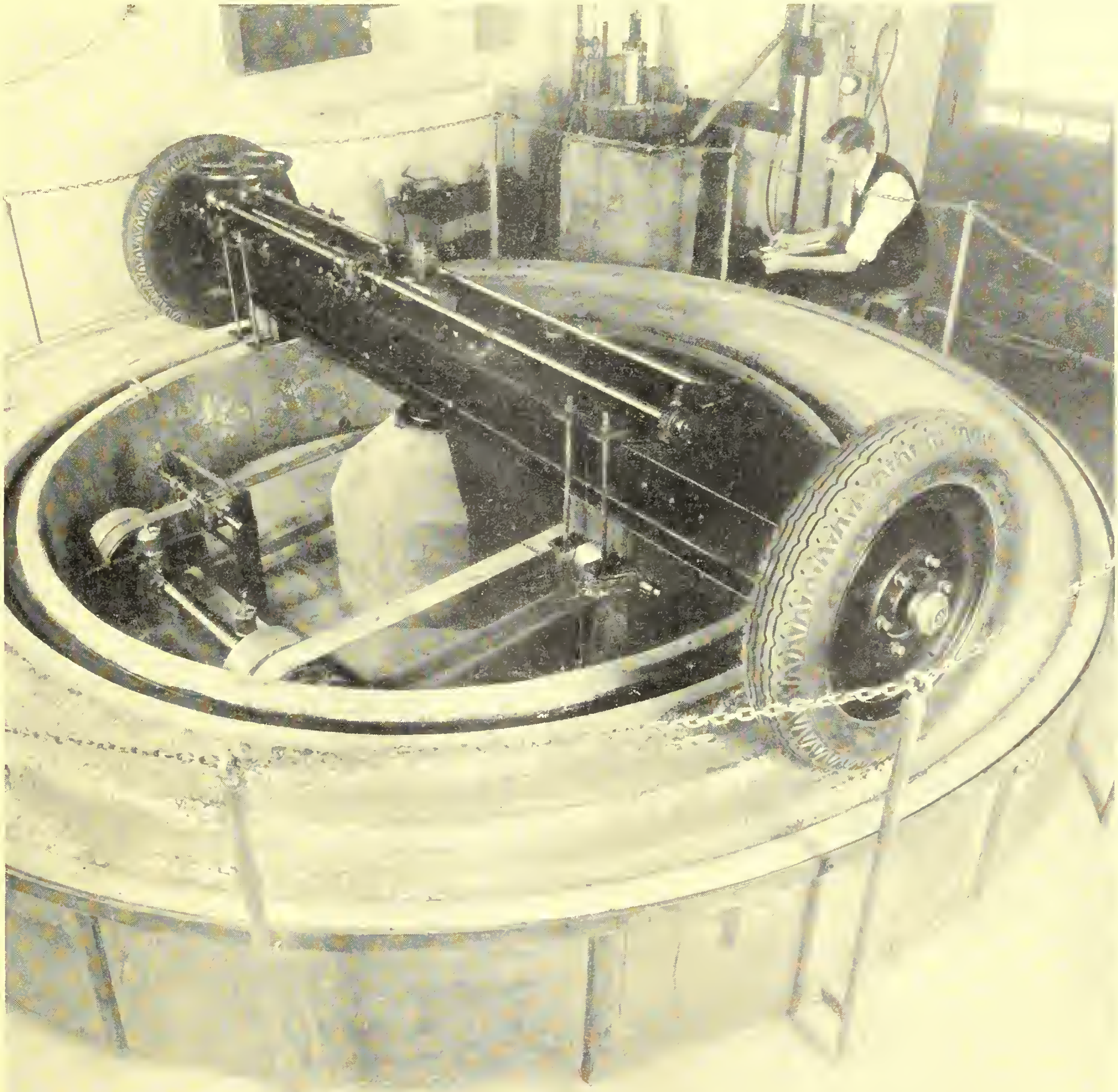
STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 153,610	$ 153,610	4		4	$ 502,248	$ 501,224	5			$ 548,900	$ 548,400	6		3	$ 782,285
Arizona						9,452	9,452									620,761
Arkansas						425,535	424,296	10			209,841	209,472	4			1,125,001
California	366,760	366,760		2		1,086,983	1,086,408	5	2		326,626	317,154	3			1,929,887
Colorado	17,980	17,979	1		1	71,588	71,588	1			25,042	21,282			10	1,177,463
Connecticut																844,490
Delaware											77,270	77,270			23	416,480
Florida						10,616	10,616				391,900	391,900	3			1,003,281
Georgia						18,346	18,346				362,690	362,690	6			2,036,351
Idaho	87,825	87,652	1			12,342	12,342				250,314	250,314	6			472,562
Illinois	186,500	186,500	1		1	1,544,675	1,544,675	5	2		660,230	660,230	6		60	2,813,238
Indiana	314,886	228,900	1	3		555,600	528,700	4	2		689,780	689,780	1	1	89	966,769
Iowa	527,765	498,900	4		9	694,219	653,600	7	1	5	48,001	44,400		2	7	1,365,724
Kansas	326,940	326,940	7		4	506,321	506,321	6			360,123	360,123	6		7	1,781,124
Kentucky	145,000	145,000	1			51,100	51,100	1			548,746	548,746	5	2	31	1,069,199
Louisiana						225,285	196,478	3			248,872	248,090	9			1,132,503
Maine	48,590	48,590	2			327,315	327,315	3	2		2,610	2,610			1	312,688
Maryland						64,586	64,586	1								962,247
Massachusetts						122,260	118,859		2		279,660	279,550	1	1		1,514,879
Michigan	393,000	393,000	5	1	7	912,458	912,458	6		38	238,600	238,600	3	1		1,741,127
Minnesota	40,218	40,218		1	1	770,180	770,180	3	5							1,614,049
Mississippi						355,700	355,700	4			363,800	363,800	3			864,651
Missouri	128,120	128,120	2	1		223,970	223,970	2			380,320	380,320	2			2,292,843
Montana	248,112	243,230	3			569,521	569,521	5								511,720
Nebraska	29,220	29,220	1			296,443	296,443	12			497,755	497,755	5		28	940,992
Nevada	97,187	97,187		2	2						57,561	57,561		1	2	303,893
New Hampshire	61,732	61,425		1		7,406	7,406				86,081	86,022	5			338,897
New Jersey						210,005	204,778	2	1		130,800	130,800		1	2	1,634,678
New Mexico						200,118	200,118	4	1		128,355	128,355	3			523,018
New York	141,400	141,400		1	2	2,472,972	2,417,071	9	7		214,000	183,150		2		4,028,308
North Carolina	73,550	73,550		1		470,810	470,810	5	1		608,330	575,630	3	4	11	1,324,404
North Dakota						606,278	606,278	2	2							972,943
Ohio						103,830	103,830	2			465,700	465,700	5			3,659,638
Oklahoma						17,343	17,343				52,075	52,075			14	2,212,091
Oregon	95,263	94,614		2		220,687	220,687	1			51,075	51,075				608,955
Pennsylvania	138,043	122,837	1			161,073	161,073	1			200,021	191,000	1			5,257,625
Rhode Island						314,484	314,484	1	2		20,535	20,535		1		158,731
South Carolina						86,379	63,593	1	2		272,740	222,140	12		34	1,203,369
South Dakota	7,530	7,530			2	316,708	314,568	2	2	7	105,850	105,850	5		1	891,343
Tennessee						14,381	14,381				87,380	87,380		2	1	1,790,272
Texas	36,660	35,990	2			928,672	928,672	8	2		389,767	389,155	5	1		3,987,972
Utah	1,760	1,760			1	142,818	142,818	4			5,319	5,319			3	423,166
Vermont	202,882	197,882	6	1		27,454	27,454		1		24,470	24,470			9	230,321
Virginia	245,670	245,670	12	1		241,018	241,018	3	1		304,555	304,555	3		8	1,004,787
Washington	149,718	147,618	1	2	2	613,457	613,457	7	2		31,117	31,117			7	591,609
West Virginia	120,100	119,080	1			291,909	291,909	3			60,870	60,870	2		5	852,239
Wisconsin	100,586	100,586	1			1,110,736	1,068,297	10			210,283	155,538	1			1,149,088
Wyoming						161,544	161,544		2	1	12,410	12,410			5	502,776
District of Columbia											34,262	34,262		1		291,169
Hawaii						33,230	33,230	1			152,570	152,570	2	1		307,950
Puerto Rico						110,940	110,390	3			116,096	115,440	1			504,470
TOTALS	4,486,607	4,341,748	57	19	36	18,220,995	17,989,387	152	42	51	10,333,302	10,135,465	117	21	361	64,646,026

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 9 NOVEMBER 1938



CIRCULAR TRACK USED IN TESTING ROAD MATERIALS

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
D. M. BEACH, *Editor*

Volume 19, No. 9 November 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE. By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business.

A STUDY OF SAND-CLAY MATERIALS FOR BASE COURSE CONSTRUCTION

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by C. A. CARPENTER, Associate Civil Engineer, and E. A. WILLIS, Associate Highway Engineer

LONG EXPERIENCE with low-cost roads in the southern States has demonstrated that certain natural mixtures of sand and clay, with varying amounts of coarser materials, serve excellently as road surfaces for relatively light traffic. As traffic on such roads increases, the necessity for applying some type of treatment to provide abrasion resistance becomes increasingly apparent. Experience in the use of surface treatments has indicated that roads highly stable before treatment may become unstable bases when covered with a waterproof surfacing. This was clearly shown by a survey of surface treatments on sand-clay, topsoil, and gravel bases in North Carolina conducted by the Bureau of Public Roads in cooperation with representatives of the tar industry in 1932.[1]

In order to study further the properties of materials suitable for low-cost bases for bituminous wearing courses, the indoor circular track shown on the cover page was utilized to investigate the behavior of 11 different sand-clay base-course materials under controlled traffic and moisture conditions. The investigations were conducted in the Bureau's laboratories at the Arlington Experiment Farm, Arlington, Va.

This circular track, which has been fully described in PUBLIC ROADS, vol. 14, no. 11, page 219, and vol. 17, no. 4, page 69, consists of a circular concrete trough 12 inches deep, 18 inches wide, and 12 feet in diameter at the center line. Facilities are provided for introducing water into the trough through the bottom of the inner wall and for maintaining any desired water elevation in the trough, as shown in figure 1.

Traffic is applied by two automobile wheels, equipped with low-pressure tires and mounted on the ends of a centrally pivoted steel beam. The tires used in these tests were of the balloon type, size 6.00 20, inflated to 35 pounds per square inch. The normal load on each wheel consisted of one-half the weight of the beam and wheel assembly, or 800 pounds. This was increased to 1,000 pounds during the later stages of the tests by hanging lead weights on the beam.

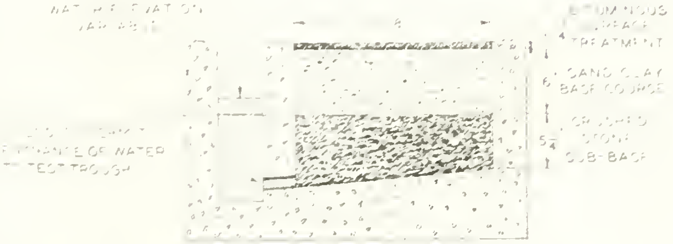

WATER ELEVATION VARIABLE
8"
BITUMINOUS SURFACE TREATMENT
6" SAND-CLAY BASE COURSE
$5\frac{3}{4}$" CRUSHED STONE SUB-BASE
ENTRANCE OF WATER TO TEST TROUGH

FIGURE 1.—CROSS SECTION OF TRACK SHOWING ARRANGEMENT OF MATERIALS FOR TESTING SAND-CLAY BASE COURSES.

Distributed traffic, which was used for compacting and in the early stages of the tests, was obtained by gradually shifting the axis of rotation of the beam, causing the wheels to pursue an alternately expanding and contracting spiral course covering the entire track area. Concentrated traffic, which was used in the later stages of the tests, was obtained by locking the sliding pivot of the beam in such a position that the wheels pursued two concentric circular courses whose center lines were about 2½ inches on either side of the center line of the test sections.

The 11 sand-clay materials used in the tests were made up of Potomac River sand, pulverized silica, and a red clay soil of local origin having a liquid limit of 90 and a plasticity index of 78. These materials were combined in various proportions to produce mixtures having the gradings and plasticity indexes shown in table 1. Five of these mixtures, referred to in table 1 as series 1, were designed to have essentially the same grading and to have plasticity indexes varying from 0 to 18. The grading selected was similar to that of materials used extensively in actual sand-clay base construction. The other six mixtures, referred to in table 1 as series 2, were designed to have a plasticity index of approximately 5 and various gradings.

The proportions of sand to clay required to produce the desired plasticity indexes were determined by soil tests on preliminary mixtures. For this purpose, use was made of the following procedure which has been

TABLE 1.—*Gradings and soil constants of sand-clay base course materials*

	Series 1					Series 2					
	Section 1	Section 2	Section 3	Section 4	Section 5	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
Grading:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Passing No. 10 sieve	100.0	[illegible]	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Passing No. 20 sieve	91.4	[illegible]	91.7	90.0	90.6	80.2	87.7	84.1	93.8	82.9	67.3
Passing No. 40 sieve	70.6	65.6	67.7	67.1	67.3	69.2	[illegible]	64.5	80.2	50.6	47.5
Passing No. 100 sieve	[illegible]	[illegible]	[illegible]	33.4	33.7	[illegible]	37.7	26.0	[illegible]	25.0	33.8
Passing No. 200 sieve	25.1	[illegible]	[illegible]	28.6	29.1	44.7	[illegible]	[illegible]	40.8	20.2	29.3
Passing 0.005 mm	10.5	[illegible]	[illegible]	20.7	21.5	15.0	13.0	[illegible]	16.0	12.0	12.6
Passing 0.001 mm	6.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	8.0	[illegible]	[illegible]	8.0	8.5
Dust ratio[2]	36	[illegible]	40	43	43	[illegible]	48	31	[illegible]	40	62
Tests on material passing No. 40 sieve:											
Liquid limit	[illegible]	[illegible]	[illegible]	28	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	20	20
Plasticity index	[illegible]	[illegible]	[illegible]	[illegible]	18	4	4	[illegible]	6	6	4

[2] Dust ratio $=100\left[\frac{\text{percentage passing No. 200 sieve}}{\text{percentage passing No. 40 sieve}}\right]$.

---

[1] Tar Surface Treatment of Low Cost Roads. PUBLIC ROADS, vol. 14, no. 1, March 1933.

found advantageous in proportioning available materials for use in actual construction.

PROPORTIONS OF MATERIALS ESTABLISHED BY PRELIMINARY TESTS

Plasticity tests were made on a number of mixtures which contained different amounts of the clay soil. The liquid limits and plasticity indexes were plotted against the clay soil contents of the mixtures, as shown in figure 2. The proportions of clay soil necessary to give the mixtures the desired plasticity indexes were then determined from the lower curve. The upper curve showed the corresponding liquid limit. In series 1, where a constant grading was desired, various proportions of fine clay soil and pulverized silica, both passing the No. 200 sieve, were used to vary the plasticity index without materially altering the total amount passing the No. 200 sieve.

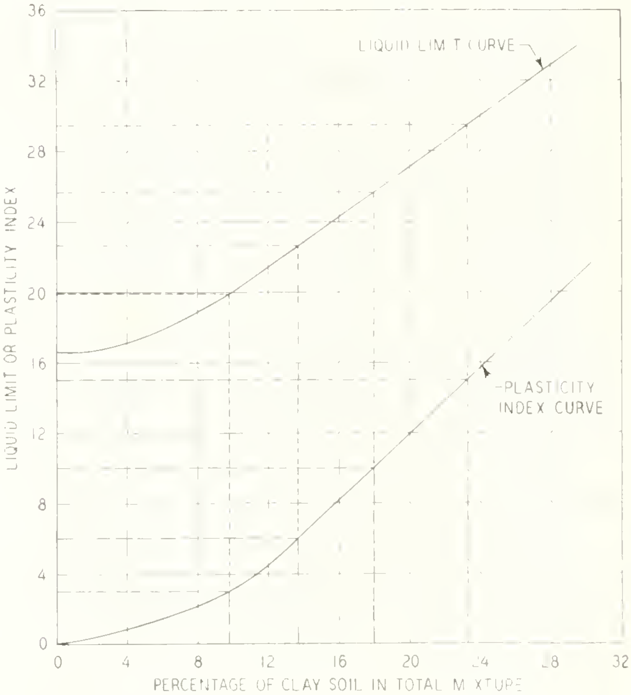


Figure 2.—Effect of Clay Soil Content on the Liquid Limit and Plasticity Index of Sand-Clay Mixtures.

The gradings and soil constants given in table 1 are those of the final mixtures as prepared for the circular track tests.

All five of the mixtures in series 1 were tested simultaneously in the track, each mixture comprising a test section 18 inches wide and approximately 7.5 feet long. In the second track containing the six mixtures of series 2, the length of each test section was approximately 6.3 feet, or one-sixth of the track circumference. The depth of each test section was approximately 6 inches after compaction.

In constructing track 1, the materials for the five test sections were combined with water to attain approximately 1 percent more than their optimum moisture contents as determined by the Proctor compaction test,[2] while the six materials in track 2 had slightly less than their Proctor optimum moisture contents. This difference was introduced for two reasons: First, because the loss of water during mixing under laboratory conditions proved to be less than was anticipated; and second, because the track tests of series 1 and concurrent molding tests utilizing direct compression of 3,000 pounds per square inch indicated that higher densities could be produced than had been obtained by the Proctor method. It was also found that maximum density was obtained both by direct compression and in the track at lower moisture contents than by the Proctor method.

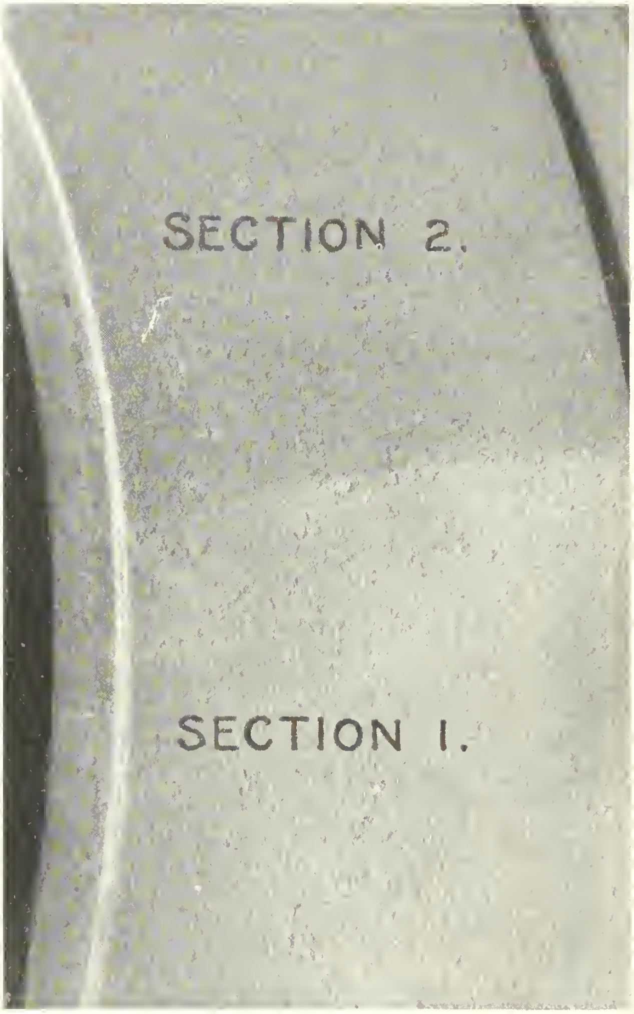


Figure 3.—Typical Appearance of Sand-Clay Base Sections in Circular Track After Compacting and Trimming Preparatory to Applying Surface Treatment.

In actual road construction, the slight excess of moisture resulting from the use of the Proctor optimum moisture content is advantageous in insuring that sufficient moisture for compaction will remain after the necessarily longer period of mixing.

The optimum moisture contents as determined by the Proctor tests and the moisture contents of the track sections after the first day's compacting traffic are given in table 2.

Aside from the differences noted in moisture contents, the procedure for preparing the materials for the track tests, constructing the test sections, and surface treating them, was the same for both tracks and was as follows:

1. The moistened sand-clay mixtures were first thoroughly mixed to distribute the water uniformly and

---

[2] Fundamental Principles of Soil Compaction, by R. R. Proctor, Engineering News-Record, vol. 111, nos. 9, 10, 12, and 13

TABLE 2.—*Optimum moisture cantents and actual moisture contents af track sectians after 1 day af compaction by traffic*

Test section	Proctor optimum moisture content	Moisture in sections after 1 day of compaction
Series 1	*Percent*	*Percent*
Section 1	8.6	8.3
2	9.3	10.0
3	9.5	10.7
4	10.3	11.0
5	10.6	11.7
Series 2		
Section 1	10.0	8.3
2	9.8	7.3
3	10.2	8.7
4	9.9	8.3
5	10.0	8.6
6	9.4	6.9

were then placed in the track in two approximately equal layers, each layer being compacted with pneumatic-tired traffic uniformly distributed over the surface.

2. Compaction was continued on the top layer until no further subsidence was noted, 8,000 wheel-trips being applied to the sections in series 1 and 32,000 wheel-trips to the sections in series 2.

3. The sections were trimmed smooth as shown in figure 3 and allowed to dry for several days.

4. A prime coat consisting of 0.3 gallon per square yard of light tar was applied and allowed to cure.

5. A surface treatment consisting of an application of 0.4 gallon of hot application bituminous material and a cover of 50 pounds per square yard of ¾-inch maximum size stone was constructed.

6. The treatment was consolidated by additional distributed traffic until the surface was well sealed and showed no movement.

Series 1.—The schedule of traffic application and changes in water elevation, with notations on the behavior of the five test sections of series 1, are given in table 3. Table 4 shows the densities and the moisture and void contents of the track sections at successive stages during compaction and at the conclusion of the traffic test.

The data in table 4 were obtained from tests on core samples taken with a 2-inch tubular core cutter. All holes from which samples were taken were filled immediately with excess material from the original mixtures, thoroughly tamped into place by hand. The factors of density and moisture content were not investigated during the test period from 36,000 wheel-trips to 266,000 wheel-trips because it was believed that sampling during this time would cause premature failures in the vicinity of the places where samples were taken. The data in table 4 represent the average conditions in the top half of the base-course sections. In series 1, the bottom half of each section was less dense than the top half and contained more water late in the compaction period and at the conclusion of the test.

ADMITTING WATER TO BASE CAUSED EARLY FAILURE OF SOME SECTIONS

Figure 4 shows the rate of consolidation and the rate at which vertical displacement occurred in the track sur-

TABLE 3.—*Schedule of operations and behavior of test sections in circular track tests, series 1*

Operation	Traffic	Water level above top of sub-base	Behavior					Remarks
			Section 1, P. I.=0	Section 2, P. I.=5	Section 3, P. I.=9	Section 4, P. I.=13	Section 5, P. I.=18	
	Wheel-trips	*Inches*						
Compacting base	0–8,000	(1)	Good	Unstable	Slightly unstable	Unstable	Good	
Compacting base and surface treatment.	[2] 8,000–36,000	(1)	do	Slightly unstable	do	Slightly unstable	do	
Testing with distributed traffic.[3]	36,000–120,000	0	do	do	Good	do	Failed	Air trapped above water under impervious surface seal.
Testing with concentrated traffic.	120,000–160,000	0	do	Good	do	do		Do.
Do	160,000–176,000	0	do	do	do	Unstable, failed		Trapped air released allowing water to rise.
Do	176,000–248,000	+2	do	do	Some movement			
Do	248,000–266,000	+3	do	do	Some cracking			Tests discontinued.

[1] No water in sub-base.
[2] Water admitted to sub-base at 36,000 wheel-trips.
[3] Wheel loads increased from 800 pounds to 1,000 pounds at 107,000 wheel-trips.

TABLE 4.—*Water content and density af circular track sections from tests on core samples* [1] *in series 1*

Number of wheel-trips when core samples taken	Condition of sub-base	Section 1				Section 2				Section 3				Section 4				Section 5			
		Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume		
			Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
2,200	Dry	8.3	15.8	70.8	13.4	10.0	19.8	73.7	6.5	10.7	21.4	74.6	4.0	11.0	21.7	73.5	4.8	11.7	23.1	73.6	3.3
8,000	do	5.8	11.0	70.8	18.2	8.3	17.4	78.0	4.6	9.6	19.8	76.8	3.4	9.5	19.5	76.5	4.0	10.5	21.2	75.3	3.5
20,000	do	4.2	8.0	70.9	21.1	6.9	15.0	81.1	3.9	7.8	16.5	79.1	4.4	8.6	18.2	79.0	2.8	9.3	19.2	77.1	3.7
36,000	Water admitted					6.7	14.7	81.6	3.7	7.7	16.5	79.7	3.8	9.0	19.2	79.6	1.2				
[2] 266,000	Wet	7.6	15.3	74.8	9.9	6.7	14.5	80.8	4.7	8.0	17.1	79.5	3.4	8.9	18.6	78.1	3.3	10.3	21.0	76.2	2.8

[1] All core samples were taken from tbe top half of the base course.
[2] All traffic up to 120,000 wheel trips distributed; traffic from 120,000 to 266,000 wheel trips concentrated.

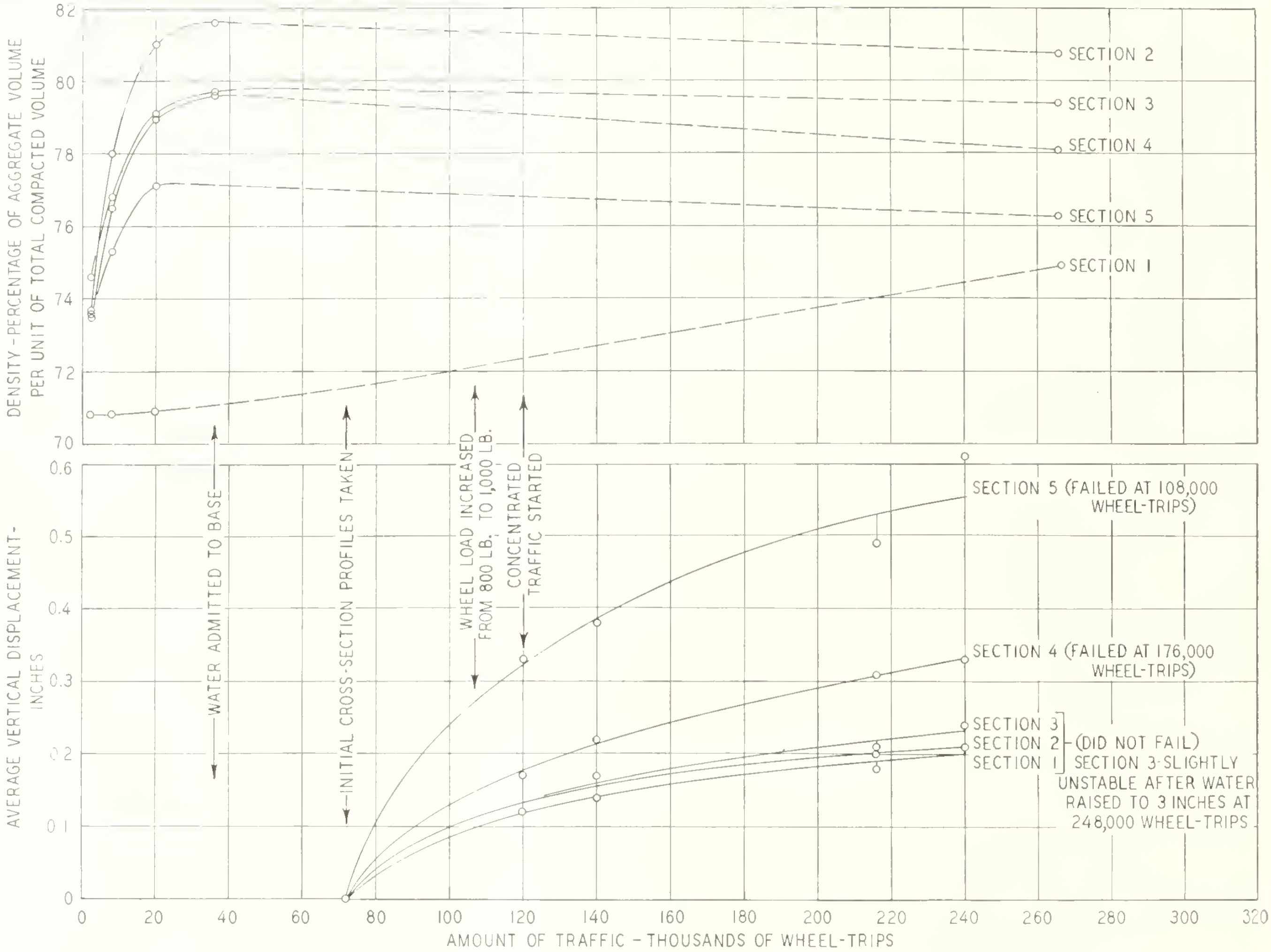


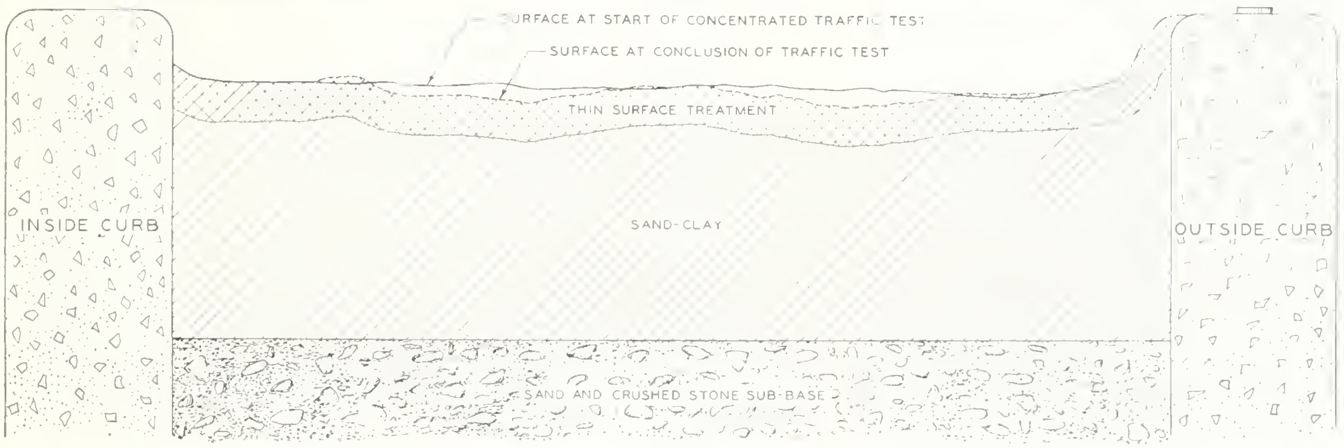
Figure 4.—Rates of Consolidation and Surface Displacement Under Traffic, Series 1.

face under traffic. The upper set of curves, showing rate of consolidation, was plotted from the data in table 4. The lower set, showing rate of surface displacement, was plotted from data obtained by means of a recording transverse profilometer with which changes in shape and elevation of the surface caused by displacement of material within the base course were measured. Typical examples of the records obtained with the transverse profilometer are shown in figure 5. Two stations were established on each test section for taking periodic readings with the profilometer and each point on the displacement curves in figure 4 represents the average of the displacement obtained at the two stations.

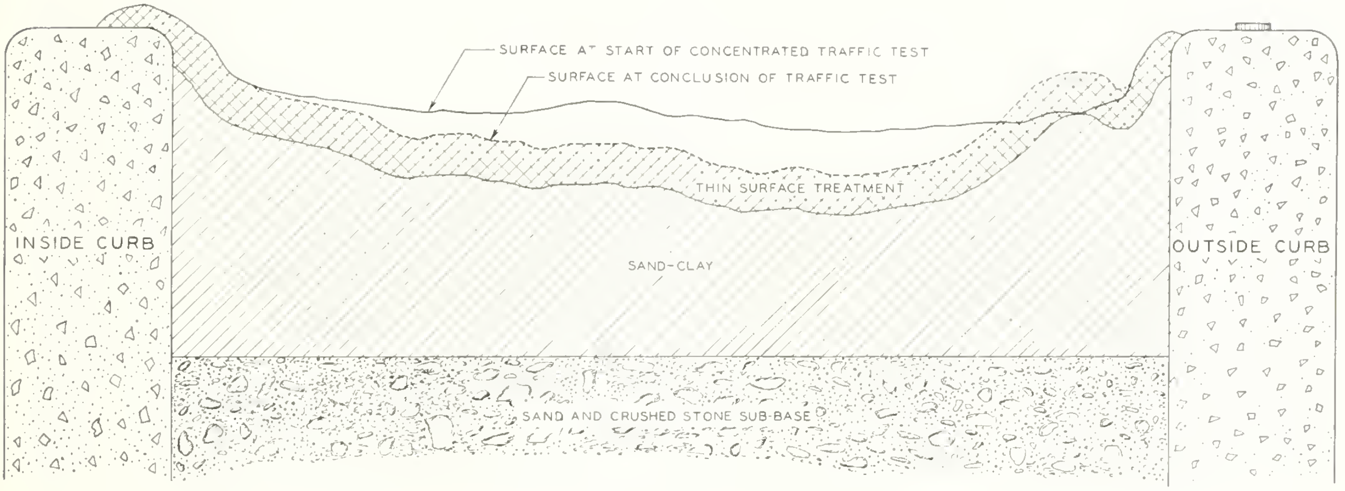
The comparative quality of the various materials used in both series 1 and series 2 was judged primarily by the amount of vertical displacement produced in the test sections by the test traffic and the rate at which this displacement occurred. It was observed that, regardless of the time required to produce it, an average vertical displacement of about 0.25 inch was sufficient to cause marked damage to the bituminous wearing course. Such damage consisted of cracking, curling, and separation of the surface treatment from the base course. It was also observed that the rate of displacement was relatively high early in the test period (see figure 4) and became progressively less as traffic was continued. That there was a wide difference in the quality of the materials in both tracks was shown by the fact that the satisfactory materials withstood from 90,000 to 140,000 more wheel-trips than the unsatisfactory materials without showing appreciable indications of failure.

In series 1, section 4 failed at approximately 176,000 wheel-trips, or 140,000 wheel-trips after water was admitted to the sub-base. Previous to that time or at approximately 108,000 wheel-trips, section 5 had failed. Sections 1 and 2 continued to withstand traffic without any indication of failure to the end of the test at 266,000 wheel-trips. Section 3 remained in good condition, except for minor cracking, up to 266,000 wheel-trips but could be seen to move slightly under each wheel passage after the water level was raised to 3 inches above the bottom of the base course at 248,000 wheel-trips. The material in section 3 was therefore classed as doubtful or border-line.

The behavior of the test sections in series 1 is well illustrated by the appearance of sections 1, 4, and 5 during the traffic test, as shown in figure 6. Sections 4 and 5 had definitely failed as indicated by excessive movement, rutting and corrugation, and the breaking and scaling of the surface treatment. Although section 1 looked slightly better at this stage than sections 2 and 3, the difference was so slight that the left illustration in figure 6 may be considered typical of the appearance of sections 1, 2, and 3 at this time and also at the conclusion of the test at 266,000 wheel-trips.


SURFACE AT START OF CONCENTRATED TRAFFIC TEST
SURFACE AT CONCLUSION OF TRAFFIC TEST
THIN SURFACE TREATMENT
INSIDE CURB
SAND-CLAY
OUTSIDE CURB
SAND AND CRUSHED STONE SUB-BASE

SECTION 1 - PLASTICITY INDEX 0


SURFACE AT START OF CONCENTRATED TRAFFIC TEST
SURFACE AT CONCLUSION OF TRAFFIC TEST
THIN SURFACE TREATMENT
INSIDE CURB
SAND-CLAY
OUTSIDE CURB
SAND AND CRUSHED STONE SUB-BASE

SECTION 5 - PLASTICITY INDEX 18

FIGURE 5.—TYPICAL CROSS SECTIONS OF TEST TRACK SHOWING COMPARATIVE EFFECTS OF CONCENTRATED TRAFFIC.

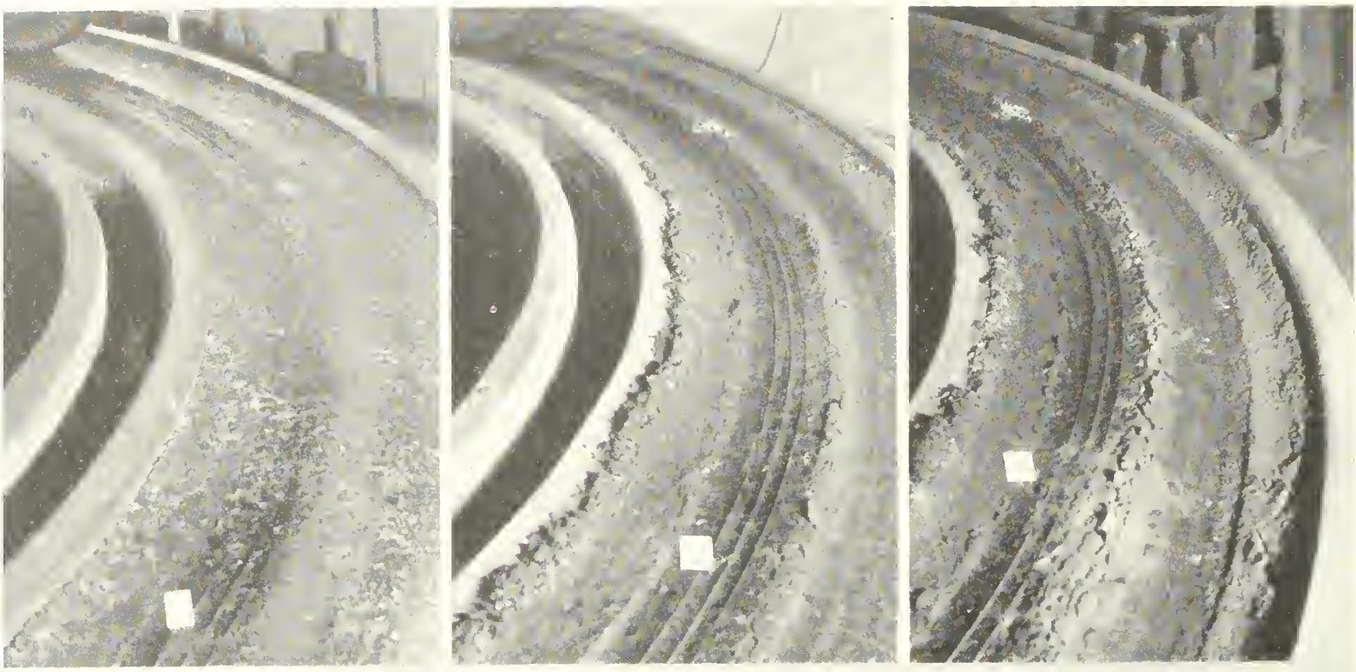
Series 2.—The schedule of traffic application and changes in water elevation with notations on behavior of the test sections in series 2 are given in table 5. Table 6 shows the densities and the moisture and void contents in the top half of the track sections during the compaction period and at the conclusion of the test. Data for sections 1, 2, and 4 are given up to the time of failure. These sections failed completely almost immediately after the water was admitted to the sub-base and had to be replaced with more stable material before the test on the three remaining sections could be continued.

Figure 7 shows the rates of consolidation and vertical displacement for the sections in series 2. The upper group of curves, showing rate of consolidation, was plotted from table 6 and the lower group showing rate of displacement was plotted from the cross section profile data obtained in the same way as that described in connection with series 1.

TABLE 5.—*Schedule of operations and behavior of test sections in circular track tests, series 2*

Operation	Traffic	Water level above top of sub-base	Behavior					
			Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
	Wheel-trips	*Inches*						
Compacting base	0–32,000	(1)	Good	Good	Unstable	Good	Unstable	Slightly unstable.
Compacting base and surface treatment.	[2] 32,000–60,000	(1)	do	do	Good	do	Good	Good.
Testing with distributed traffic [3]	60,000–120,000	+½	Failed at 61,000	Failed at 63,000	do	Failed at 61,000	do	Do.
Testing with concentrated traffic	120,000–200,000	+½			do		do	Slightly unstable.

[1] No water in sub-base.
[2] Water admitted to sub-base at 60,000 wheel-trips.
[3] Wheel loads increased from 800 pounds to 1,000 pounds at 82,000 wheel-trips.



SECTION 1. 176,000 WHEEL-TRIPS SECTION 4. 176,000 WHEEL-TRIPS SECTION 5. 140,000 WHEEL-TRIPS

FIGURE 6.—APPEARANCES OF THREE SECTIONS OF TRACK 1 AFTER VARIOUS AMOUNTS OF TEST TRAFFIC. SECTIONS 4 AND 5 HAD FAILED.

TABLE 6.—*Water content and density of circular track sections from tests on core samples [1] in series 2*

Number of wheel-trips when core samples taken	Condition of sub-base	Section 1				Section 2				Section 3				Section 4				Section 5				Section 6			
		Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume			Water content based on dry weight	Composition by volume		
			Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids		Water	Aggregate	Air voids
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
2,700	Dry	8.3	15.5	70.1	14.4	7.3	14.3	73.3	12.4	8.7	17.4	74.7	7.9	8.3	16.0	72.0	12.0	8.6	17.6	76.4	6.0	6.9	14.0	76.2	9.8
24,000	do	7.5	14.3	71.3	14.4	6.6	12.9	73.3	13.8	7.9	16.9	79.7	3.4	7.5	14.5	72.6	12.9	7.5	15.7	78.4	5.9	5.8	12.4	80.0	7.6
60,000	Water admitted																								
61,000	Wet	10.5	20.9	74.7	4.4									10.4	20.8	74.9	4.3								
63,000	do					9.6	19.5	76.1	4.4																
[2] 200,000	do									7.4	15.8	79.6	4.6					5.9	12.8	81.2	6.0	5.5	11.8	80.6	7.6

[1] Core samples taken from top half of the base course.

[2] All traffic up to 120,000 wheel-trips distributed; traffic from 120,000 to 200,000 wheel-trips concentrated.

As in series 1, the displacement curves clearly distinguished the satisfactory materials from those that failed. Sections 1 and 4 withstood only 1,000 wheel-trips of distributed traffic after the water was admitted to the sub-base, and section 2 withstood only 3,000 wheel-trips. The average vertical displacements measured after failure were 0.87 inch for section 1, 0.67 inch for section 4, and 0.64 inch for section 2. Sections 3 and 5 remained in excellent condition throughout the test and showed final displacements of only 0.16 inch and 0.06 inch, respectively. Section 6 was generally stable throughout the test and showed a final displacement of only 0.17 inch, but at about 120,000 wheel-trips it developed sufficient movement under traffic to cause some cracking and edge raveling. Later in the test this condition disappeared but the temporary weakness shown was held to justify the classification of the material in this section as borderline.

STABILITY DURING COMPACTION NOT NECESSARILY AN INDICATION OF QUALITY

In considering the behavior of the 11 materials used in this investigation, particularly during the compaction period, section 1 of series 1 must be placed in a class entirely separate from the rest. It was the only non-plastic material in the group. Its compaction and stability relationships were decidedly different from those of the 10 plastic materials.

From figures 4 and 7 it is evident that section 1 of series 1 was the least compactible of any of the 11 materials. Its gain in density during the compaction period, after the initial determination at 2,200 wheel-trips, was so slight as to be of no significance whatever; and at the end of the test its density, while appreciably higher than at 2,200 wheel-trips, was less than the final density of any other section except section 1 of series 2. At no time, however, either during the compaction or test periods, did it show any evidence of instability.

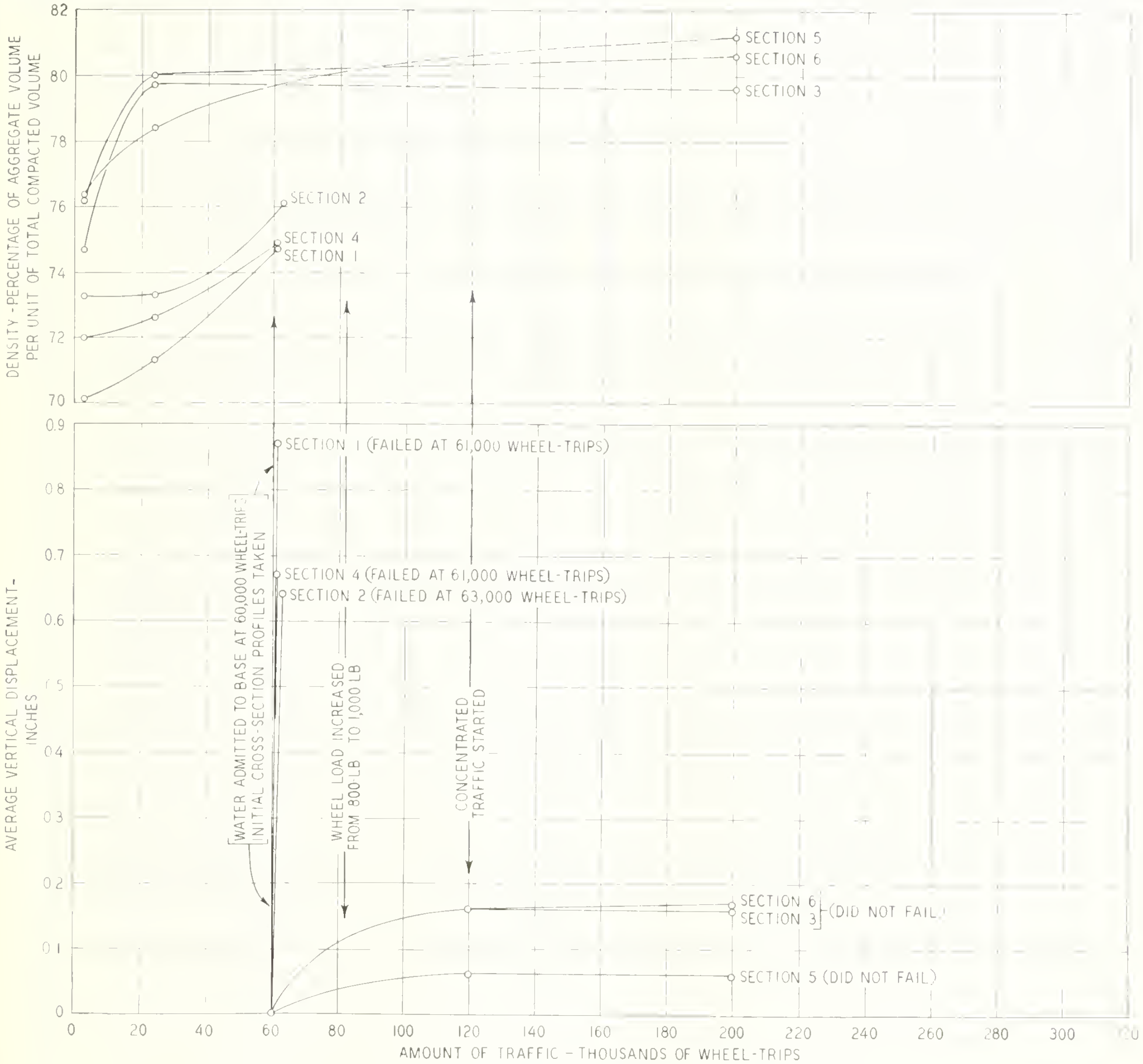


FIGURE 7.—RATES OF CONSOLIDATION AND SURFACE DISPLACEMENT UNDER TRAFFIC, SERIES 2.

Of the 10 plastic materials, the ones that gave satisfactory service in the traffic test were, without exception, definitely unstable during the compaction period. These were sections 2 and 3 of series 1, and sections 3, 5, and 6 of series 2. All these materials compacted quite rapidly and, with the exception of section 5 of series 2, reached either their highest or essentially their highest density under less than 40,000 wheel-trips of distributed traffic. They were so unstable during compaction that it was difficult to prevent them from working out over the curbs under traffic and it was only after a considerable drying period with continued traffic and periodic reshaping that they set up and stopped shoving and corrugating.

Section 4 of series 1 was unique in that it was unstable throughout both the compaction and test periods. The other four plastic materials, section 5 of series 1, and sections 1, 2, and 4 of series 2, all of which failed because of instability during the traffic test, set up early in the compaction period and required little or no reshaping. Their tendency to set up under early traffic prevented effective compaction and their densities as shown in figures 4 and 7 were lower than those of any of the satisfactory materials, except the non-plastic section 1 of series 1 previously discussed.

The lack of early stability of the materials that later proved satisfactory during the traffic test, even after prolonged exposure to capillary moisture, is emphasized for two reasons: First, because such behavior in field construction might be misconstrued and lead to the use of too much clay in order to facilitate compaction; and second, because it so clearly illustrates the importance of thorough compaction in conjunction with drying.

Section 5 of series 2, which is shown in figure 8, during early compaction on the left, and after 200,000 wheel-trips (conclusion of the traffic test) on the right, is typical of the materials that showed early lack of

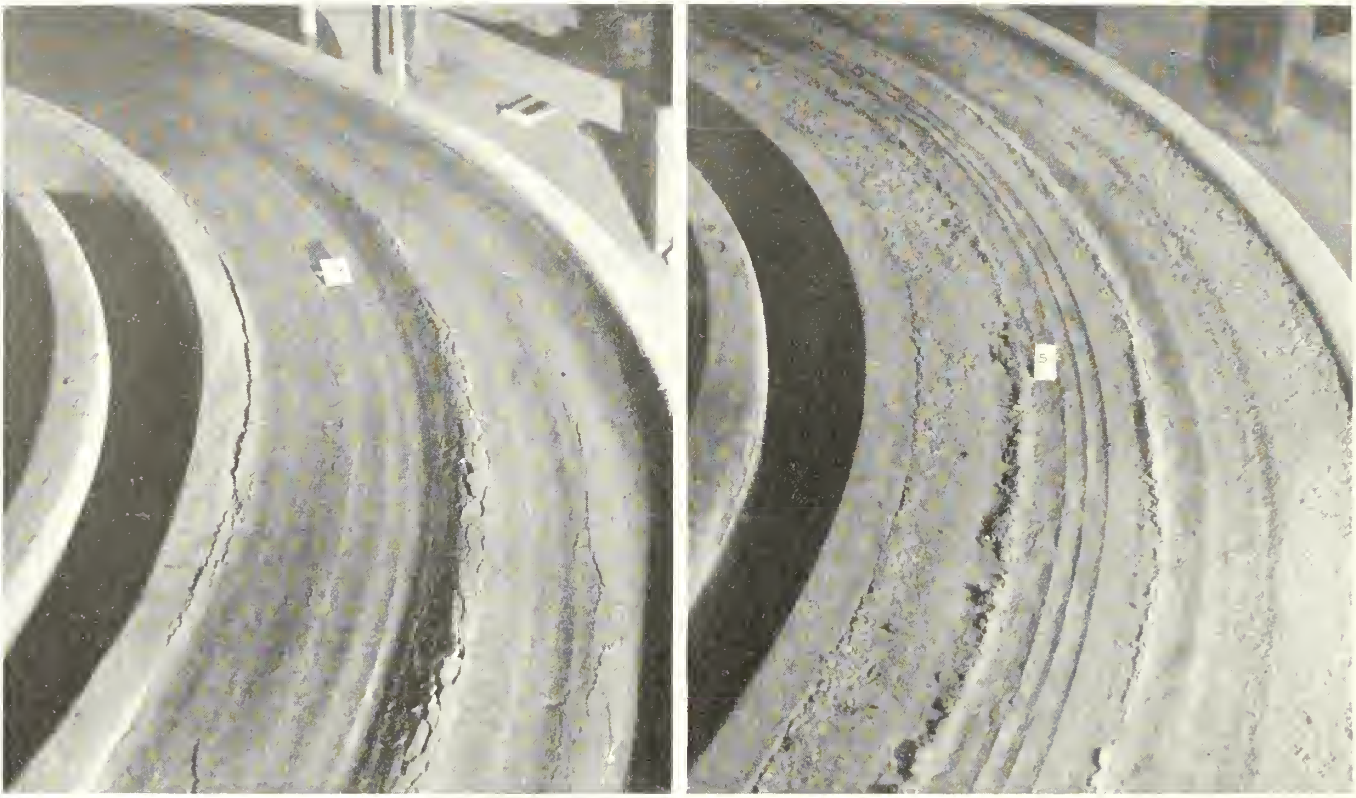


Figure 8.—Section 5 of Series 2 During Compaction at the Left and After Surface Treating and Applying Traffic to a Total of 200,000 Wheel-Trips at the Right. This Section Was Unstable During Early Compaction but Gave Excellent Service After Consolidation Was Completed.

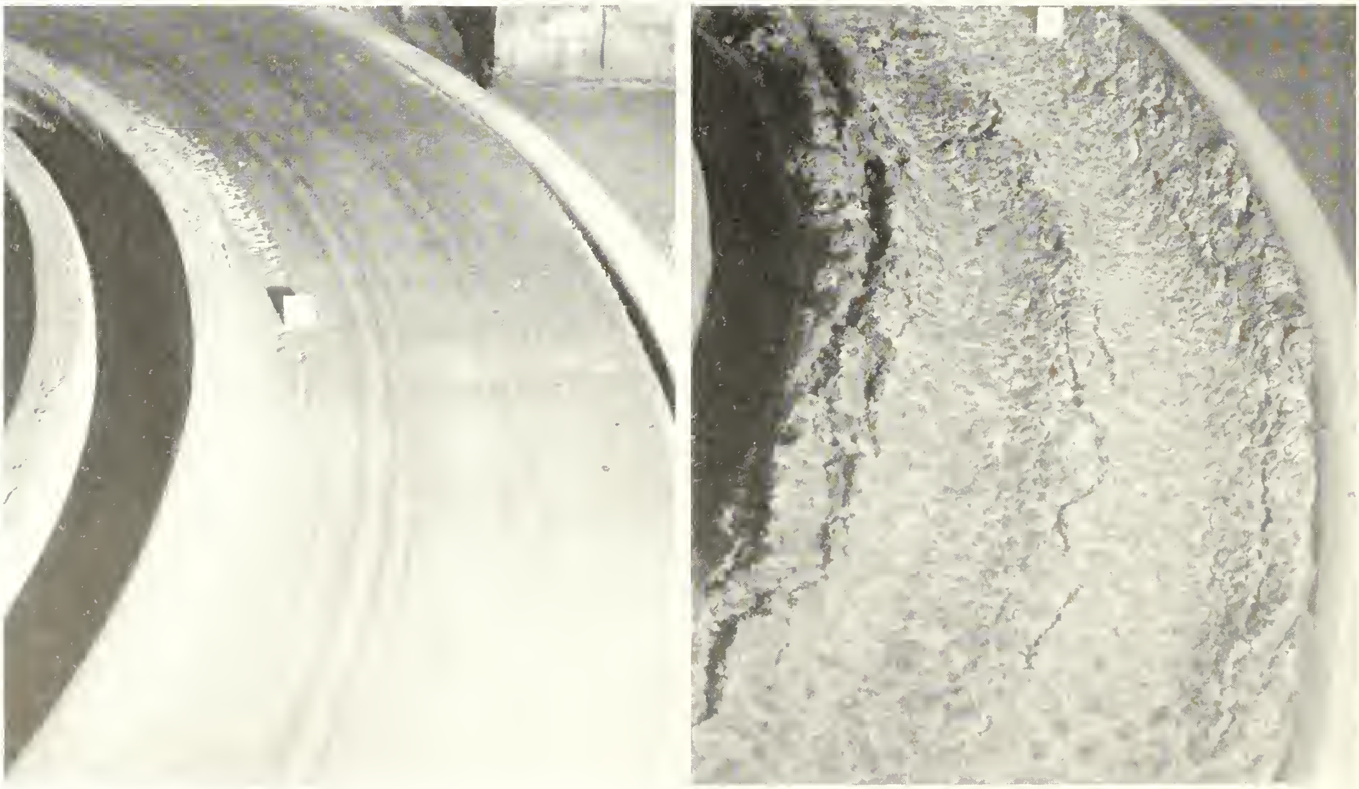


Figure 9.—Section 1 of Series 2. This Section Set Up Early During the Compaction Period, but Softened and Failed Because of Instability at 61,000 Wheel-Trips or Only 1,000 Wheel-Trips After Water Was Admitted to the Sub-base. Left Picture Taken at 20,000 Wheel-Trips or Early in Compaction Period; Right Picture Shows Surface Treated Base Section at Time of Failure.

stability and later developed high stability and resistance to the softening action of water. Figure 9 shows comparable views of section 1 of series 2, the behavior of which was typical of the sections that set up early in the compaction period but became unstable to the extent of failure under test traffic after water was admitted to the sub-base. The photograph at the left shows the section during early compaction and the one at the right shows the same section after it failed at 61,000 wheel-trips, or only 1,000 wheel-trips after water was admitted to the sub-base.

TRACK DENSITIES OF SATISFACTORY SECTIONS GENERALLY CLOSE TO MAXIMUM VIBRATED DENSITIES

Compaction tests made on the oven-dried materials from both tracks, using a vibratory method of compaction, gave the densities shown in table 7. The final densities of the track sections from tables 4 and 6 are repeated here for convenience of comparison. The densities obtained in the standard Proctor tests are also shown.

TABLE 7.—*Densities*[1] *of track materials after compaction by different methods*

Identification	Compacted by vibration	Top half of track at end of test	Proctor compaction
Series 1	*Percent*	*Percent*	*Percent*
Section 1	80.6	74.8	74.2
2	80.8	80.8	76.2
3	79.6	79.5	76.2
4	78.5	78.1	75.1
5	76.3	76.2	74.5
Series 2			
Section 1	79.6	74.7	75.0
2	81.9	76.1	76.5
3	80.5	79.6	75.3
4	78.8	74.9	75.9
5	81.4	81.2	76.2
6	82.3	80.6	77.8

[1] Percentage of aggregate volume per unit of total compacted volume.

It is believed that the densities obtained by vibrating the dry materials represent closely their maximum obtainable densities. Except for section 1 of series 1 which, because of its harshness, was stable even at the lowest density recorded, the materials that failed to compact in the track to essentially their maximum vibrated densities failed to withstand the traffic test. These were the materials in sections 1, 2, and 4 of series 2, and their lack of compactibility under traffic is attributed to their high content of material passing both the No. 40 and No. 200 sieves.

It is unlikely that the compactibility of materials 4 and 5 of series 1 played any part in their failure. They were very close to their maximum vibrated densities when failure occurred but both had sufficiently high plasticity indexes to cause them to soften and become plastic when subjected to wetting, even when fully compacted.

Except for materials 1, 2, and 4 of series 2, the densities obtained in the standard Proctor test were less than those of the track sections. However, the use of the standard Proctor test for predetermining probable actual densities of traffic- or roller-compacted, granular, road-building materials is of minor importance. It may, if desired, be modified to produce densities more nearly in agreement with those obtained on the traffic-compacted track sections by increasing either the number or the force of the tamping blows, but its value lies primarily in the ease and convenience with which it can be used to determine the necessary amount of water to obtain compaction of soil in highway construction.

Supplementary tests made in the laboratory on small samples of the materials used in the track sections were:

1. The compaction tests with the Proctor apparatus, already discussed.
2. The vibratory compaction tests by which the maximum practical density of the aggregates was investigated.
3. Hubbard-Field stability tests on core samples from the track sections after the traffic test.
4. Hubbard-Field stability tests on molded specimens of each material over a range of moisture contents.
5. Hubbard-Field stability tests on molded specimens with constant water content molded under various loads to determine the effect of density on the stability values.

The Hubbard-Field stability tests were made in an effort to develop a method of predicting, by means of simple laboratory tests on small samples, the probable behavior of sand-clay, base-course materials under unfavorable service conditions. In making the test[3] a cylindrical specimen 2 inches in diameter and approximately 1 inch high is placed in a 2-inch cylinder, in the bottom of which is a plate with a 1¾-inch orifice. The specimen is forced through the orifice and the maximum load registered on the testing machine while forcing the specimen through the orifice is recorded as the stability. The test specimens may be either sections cut from core samples and tested at field density or they may be molded from loose material by the same method as that used by the Bureau in molding samples of sheet asphalt for the stability test. This method consists of placing the required amount of loose material in a 2-inch cylinder and compressing it under a load of 3,000 pounds per square inch with plungers in both ends of the cylinder so that compaction will be the same on both faces of the specimen.

EFFECT OF MOISTURE CONTENT ON STABILITY OF SPECIMENS STUDIED

The results of the Hubbard-Field stability tests on cores from all the test sections and the corresponding moisture contents, densities, and void contents, are shown in table 8. Sections 2 and 3 of series 1 and sections 3, 5, and 6 of series 2, showed high stabilities which agreed well with their service behavior. Sections 1, 2, and 4 of series 2 showed very low stabilities, and this was likewise consistent with their service behavior. The Hubbard-Field stabilities of sections 1, 4, and 5, of series 1, however, were highly inconsistent with their service behavior. The stability test values for section 1 were only about one-third those for section 2, while the service behavior of the two sections was equally good. Section 4 showed Hubbard-Field stabilities appreciably higher than those of section 1 and the values for section 5 were only slightly less than those for section 1. Yet both sections 4 and 5 failed in the traffic test.

Because of these inconsistencies it was concluded that the Hubbard-Field stability at any single water content or density could not be relied upon to furnish information as to the probable service behavior of a sand-clay, base-course material. The results of tests next to be described did, however, show that when stability tests were made on specimens covering a range

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[3] Method described in Research Series No. 1, Oct. 15, 1935, published by The Asphalt Institute.

TABLE 8.—*Water content, density, and Hubbard-Field stability of cored specimens from the track sections in series 1 and 2*

SERIES 1

Section No.	Plasticity index	Depth	Water content based on dry weight	Composition by volume			Hubbard-Field stability	Remarks
				Water	Aggregate	Air voids		
			Percent	*Percent*	*Percent*	*Percent*	*Pounds*	
1	0	Top inch	7.5	15.2	75.4	9.4	(1)	Tested at 266,000 trips (end of traffic test).
		Second inch	7.6	15.3	74.8	9.9	1,160	
		Third inch	7.6	15.1	74.1	10.8	1,060	
2	5	Top inch	6.0	13.1	81.6	5.3	(2)	Do.
		Second inch	6.7	14.6	81.0	4.4	3,500	
		Third inch	7.4	15.8	79.8	4.4	2,820	
3	9	Top inch	7.4	15.9	79.9	4.2	3,050	Do.
		Second inch	8.0	17.1	79.9	3.0	2,570	
		Third inch	8.7	18.4	78.7	2.9	1,940	
4	13	Top inch	8.1	17.2	79.2	3.6	1,770	Do.
		Second inch	8.7	18.3	78.3	3.4	1,510	
		Third inch	9.8	20.2	76.9	2.9	1,070	
5	18	Top inch	9.2	19.0	77.3	3.7	1,100	Do.
		Second inch	10.2	21.0	76.7	2.3	740	
		Third inch	11.6	23.2	74.6	2.2	440	

SERIES 2

Section No.	Plasticity index	Depth	Water content based on dry weight	Water	Aggregate	Air voids	Hubbard-Field stability	Remarks
1	4	Top inch	10.4	20.7	74.6	4.7	670	Tested at 61,000 trips (time of failure).
		Second inch	11.2	22.2	74.5	3.3	470	
		Third inch	9.9	19.8	74.9	5.3	730	
2	4	Top inch	9.5	19.3	75.9	4.8	930	Tested at 63,000 trips (time of failure).
		Second inch	10.1	20.6	76.2	3.2	640	
		Third inch	9.3	18.9	76.2	4.9	880	
3	5	Top inch	6.6	14.3	80.7	5.0	2,500	Tested at 200,000 trips (end of traffic test).
		Second inch	7.6	16.2	79.7	4.1	2,300	
		Third inch	8.0	16.8	78.5	4.7	2,050	
4	6	Top inch	9.9	20.0	75.5	4.5	660	Tested at 61,000 trips (time of failure).
		Second inch	10.8	21.6	74.9	3.5	410	
		Third inch	10.5	20.8	74.3	4.9	400	
5	6	Top inch	5.8	12.4	79.7	7.9	3,500	Tested at 200,000 trips (end of traffic test).
		Second inch	5.9	12.9	81.6	5.5	3,630	
		Third inch	5.9	13.0	82.4	4.6	3,700	
6	4	Top inch	5.2	11.3	81.2	7.5	3,210	Do.
		Second inch	5.5	11.9	80.8	7.3	3,120	
		Third inch	5.7	12.2	79.9	7.9	3,050	

[1] Top inch contained some tar prime so that comparable stability was not obtained.
[2] Top inch too brittle to permit making stability test.

of moisture contents the rate at which the Hubbard-Field stability decreased with increasing water contents, after rising to a maximum value, bore a strikingly definite relationship to the behavior of the materials in the track test.

A series of stability tests was made on specimens of the material from each track section molded at moisture contents ranging from well below that necessary to produce maximum density to as high a content as the soil would retain during molding. The specimens in series 1 were tested for density immediately after molding. They were then tested for stability and immediately weighed and oven-dried to determine their exact moisture contents. The results of these tests are shown in table 9 and figure 10.

Table 10 and figure 11 show the results of similar tests on the materials in series 2, the only difference being that, for series 2, the density and stability values were obtained from separate sets of specimens prepared and tested on different days. No special attempt was made to use identical moisture contents in these two sets of specimens.

As shown in figures 10 and 11, the satisfactory materials (sections 1, 2, and 3 of series 1 and sections 3, 5, and 6 of series 2) exhibited relatively small changes in Hubbard-Field stability within the range of moisture contents where stability was decreasing while density was increasing, whereas, the unsatisfactory materials (sections 4 and 5 of series 1 and sections 1, 2, and 4 of series 2) showed relatively large changes within the same range. All except material 1 of series 1 lost stability very rapidly after the maximum density was

TABLE 9.—*Relation of density and stability to water content in molded specimens of series 1* [1]

Section number	Plasticity index	Water content based on dry weight	Composition by volume			Hubbard-Field stability
			Aggregate solids	Aggregate voids	Air voids	
		Percent	*Percent*	*Percent*	*Percent*	*Pounds*
1	0	1.4	--------	--------	--------	1,460
		1.8	--------	--------	--------	1,550
		3.4	--------	--------	--------	1,420
		4.3	74.3	25.7	17.1	1,420
		5.4	74.7	25.3	14.5	1,480
		5.5	--------	--------	--------	1,490
		6.2	74.9	25.1	12.6	1,570
		7.1	74.7	25.3	11.1	1,440
		8.2	75.2	24.8	8.2	1,360
		9.4	75.3	24.7	5.7	1,490
2	5	4.1	78.8	21.2	12.5	3,760
		4.7	79.4	20.6	10.6	3,650
		5.2	79.8	20.2	9.1	3,640
		7.3	81.3	18.7	2.8	3,360
		7.6	81.4	18.6	2.0	3,025
		9.6	78.4	21.6	1.4	980
3	9	4.6	79.4	20.6	10.8	3,790
		6.0	80.9	19.1	6.1	3,630
		7.1	81.4	18.6	3.1	3,510
		7.3	81.4	18.6	2.7	3,390
		8.2	80.9	19.1	1.3	2,250
		9.4	79.0	21.0	1.1	1,160
4	13	3.9	79.6	20.4	12.1	4,630
		4.7	81.3	18.7	8.5	4,310
		6.0	82.3	17.7	4.5	3,330
		6.8	82.4	17.6	2.6	2,790
		8.0	81.3	18.7	1.3	1,760
		8.7	80.0	20.0	1.3	1,310
5	18	4.2	79.9	20.1	11.1	4,520
		4.5	80.7	19.3	9.6	4,340
		6.1	81.6	18.4	5.1	3,060
		7.2	82.2	17.8	2.0	2,180
		7.6	82.2	17.8	1.1	1,770
		8.4	80.9	19.1	.9	1,350
		10.6	76.6	23.4	1.7	580

[1] All specimens were molded at the water contents shown and tested immediately, using the Hubbard-Field apparatus for molding and testing.

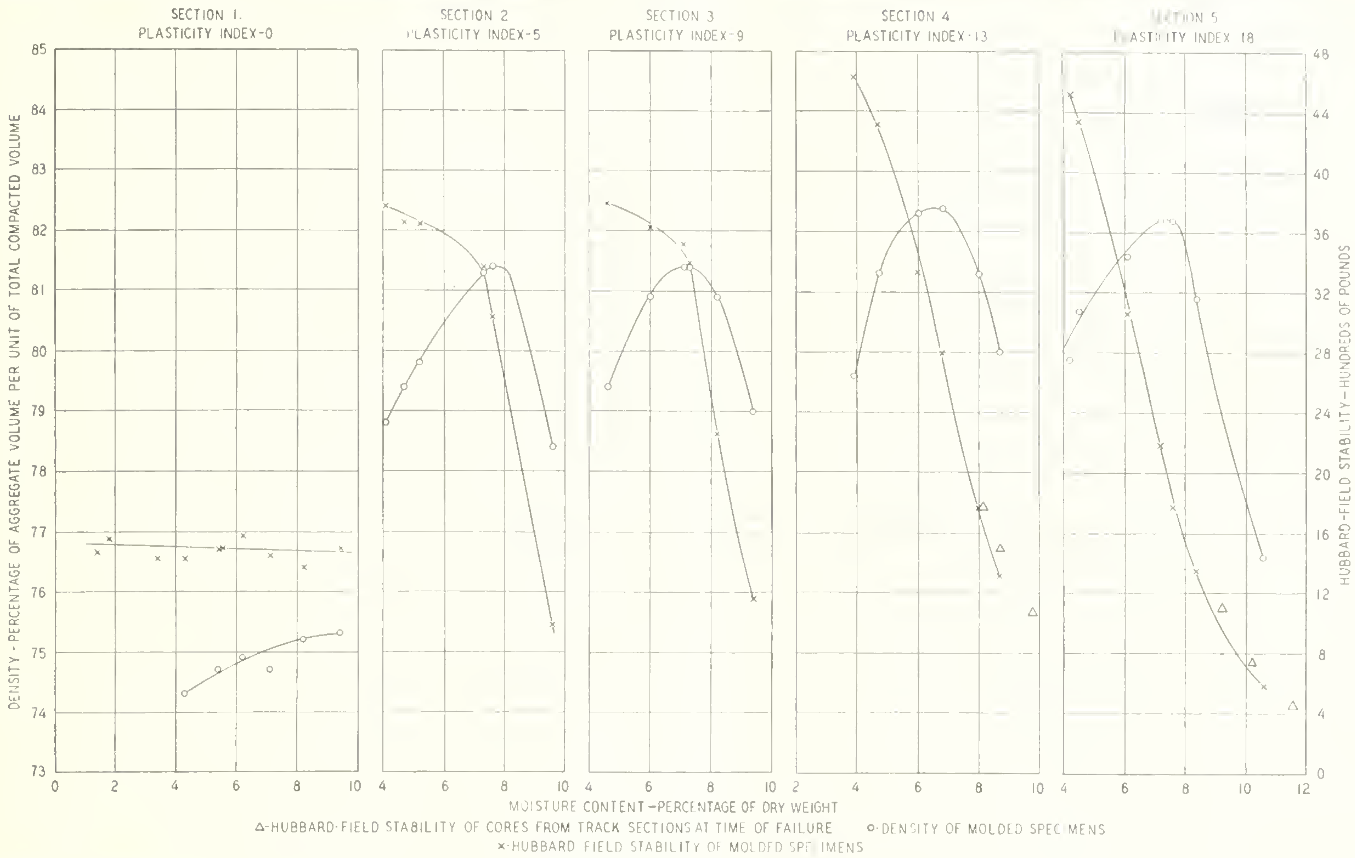

FIGURE 10.—RELATION OF DENSITY AND STABILITY TO MOISTURE CONTENT, SERIES 1. CURVES PLOTTED FROM DATA IN TABLES 8 AND 9.

TABLE 10.—*Relation of density and stability to water content in molded specimens of series 2* [1]

Section number	Plasticity index	Water content based on dry weight	Composition by volume			Hubbard-Field stability
			Aggregate solids	Aggregate voids	Air voids	
		Percent	*Percent*	*Percent*	*Percent*	*Pounds*
1	4	0.5	---	---	---	1,550
		2.6	74.2	25.8	20.7	---
		3.0	---	---	---	2,620
		5.1	75.3	24.7	14.5	---
		6.2	---	---	---	3,300
		6.8	76.8	23.2	9.3	---
		8.1	---	---	---	3,600
		8.4	77.8	22.2	4.8	---
		8.7	---	---	---	3,010
		9.0	78.3	21.7	2.9	2,310
		9.1	78.4	21.6	2.6	---
2	4	.6	---	---	---	1,730
		2.8	75.8	24.2	18.5	---
		4.2	76.3	23.7	15.1	---
		5.8	76.8	23.2	11.3	---
		6.3	---	---	---	2,900
		7.0	---	---	---	2,990
		7.9	---	---	---	2,880
		8.0	78.6	21.4	4.6	2,270
		8.8	---	---	---	1,170
		10.0	79.0	21.0	.0	---
3	5	.6	---	---	---	1,780
		3.2	74.5	25.5	19.1	---
		5.1	76.2	23.8	13.4	---
		6.3	76.7	23.3	10.4	2,200
		7.4	---	---	---	2,230
		8.3	78.2	21.8	4.4	---
		8.6	---	---	---	2,170
		9.0	79.0	21.0	2.0	2,000
		9.5	79.1	20.9	.8	---
		9.9	---	---	---	1,600
4	6	1.6	---	---	---	2,000
		2.9	74.8	25.2	19.4	---
		4.9	75.6	24.4	14.5	---
		5.8	---	---	---	3,070
		6.5	77.4	22.6	9.2	---
		7.2	---	---	---	3,530
		7.5	---	---	---	3,380
		8.1	79.3	20.7	3.5	2,990
		8.5	79.5	20.5	2.5	---
		8.7	---	---	---	1,790
		9.3	---	---	---	830

TABLE 10.—*Relation of density and stability to water content in molded specimens of series 2*—Continued

Section number	Plasticity index	Water content based on dry weight	Composition by volume			Hubbard-Field stability
			Aggregate solids	Aggregate voids	Air voids	
		Percent	*Percent*	*Percent*	*Percent*	*Pounds*
5	6	1.6	---	---	---	1,240
		2.9	74.4	25.6	19.8	1,700
		4.5	75.0	25.0	16.0	---
		6.1	75.4	24.6	12.3	1,910
		7.9	---	---	---	1,860
		8.0	---	---	---	1,900
		8.2	77.6	22.4	5.4	---
		8.9	78.5	21.5	2.8	---
		9.0	78.8	21.2	2.2	---
		9.2	---	---	---	1,690
		10.5	---	---	---	920
6	4	.5	---	---	---	1,280
		2.4	76.6	23.4	18.5	2,410
		3.3	---	---	---	2,570
		4.0	77.4	22.6	14.3	---
		5.0	---	---	---	2,800
		6.0	78.2	21.8	9.3	2,760
		6.4	---	---	---	2,710
		7.5	---	---	---	2,520
		8.0	80.2	19.8	2.7	---
		8.4	80.5	19.5	1.4	2,120
		8.8	80.0	20.0	1.2	1,990

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[1] Specimens for density and those for stability were made and tested at different times. The water contents shown were determined immediately after testing. No particular effort was made to have identical water contents in the density and stability specimens.

passed. The highest stabilities shown in figure 10 for series 1 represent only the maxima for the data obtained and are not necessarily the maxima that might have been obtained by including tests on still drier mixtures. It is obvious, however, that extension of these curves to the left through maximum obtainable stability would only serve to accentuate the difference between the good and bad materials.

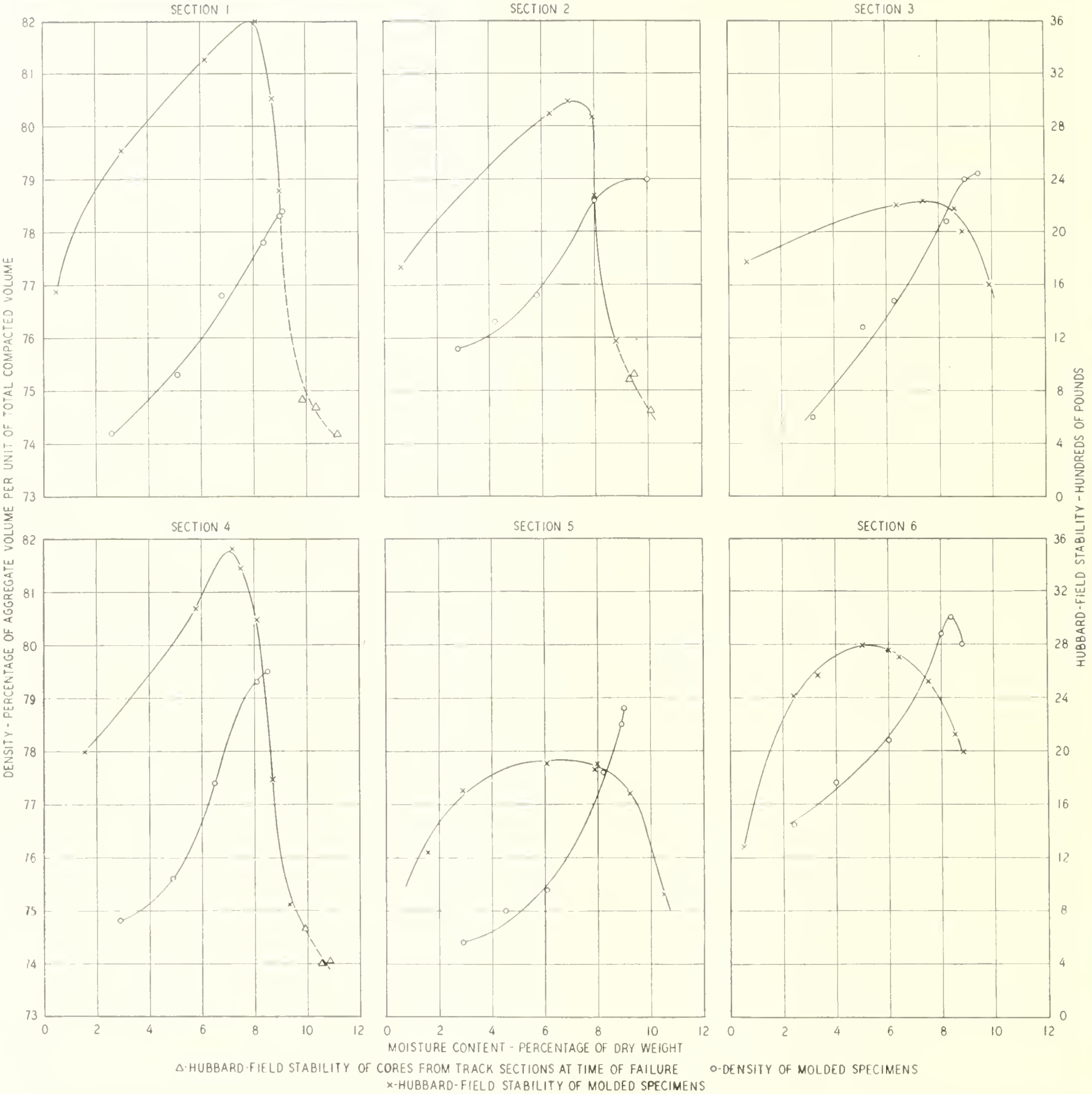


Figure 11.—Relation of Density and Stability to Moisture Content, Series 2. Curves Plotted From Data in Tables 8 and 10.

It is important to keep in mind that the changes in density and stability occurred because of changes in water content and that, in the range between maximum or essentially maximum stability and maximum density, without exception the stability was falling while the density was increasing. Tests to be described later proved that when the moisture content was held constant and density was increased by more intensive compaction, the Hubbard-Field stability invariably increased. Therefore, the losses in stability that occurred between maximum stability and maximum density (figures 10 and 11) were caused entirely by the increases in moisture content working counter to the stabilizing effect of increasing density.

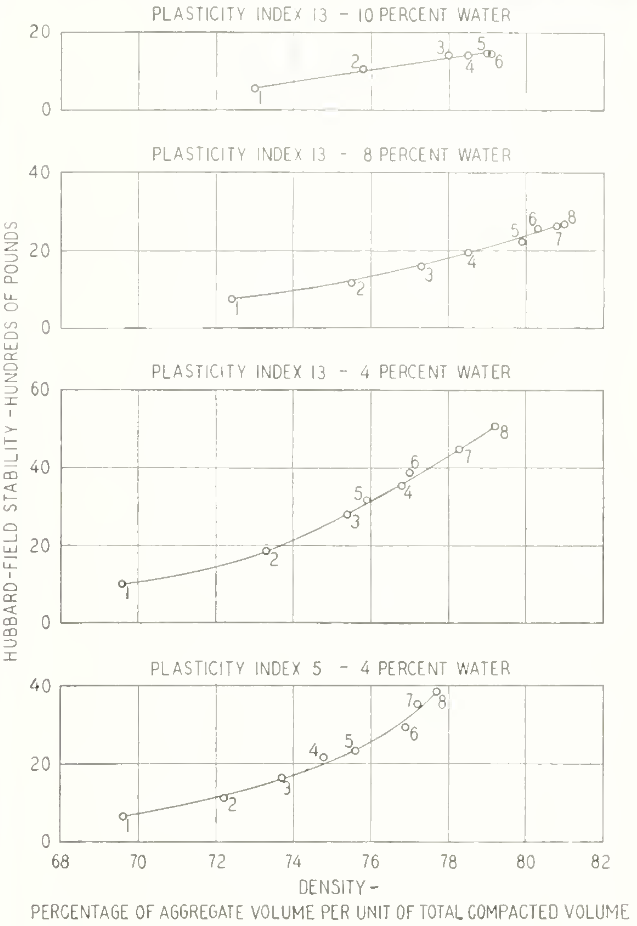
The maximum stabilities and the stabilities at maximum density, with the corresponding water contents and the mean changes in stability per percent of moisture change, are shown in table 11. These rates of change, without exception, reflect the behavior of the materials in the track tests. For the satisfactory materials, the rate of stability change was low, ranging from the negligible amount of 6 pounds per percent of moisture change for material 1 of series 1 to 229 pounds for material 3 of series 1. The unsatisfactory materials exhibited much higher rates of change, the lowest for this group being 634 pounds for material 4 of series 1, and the highest being 1,800 pounds for material 1 of series 2.

Although tests on additional materials of known service behavior should be made to establish more closely the dividing rate of stability change between good and bad materials, the described method of test appears to be fundamentally sound as a means of predicting in the laboratory the probable service behavior of fine-grained base-course or subgrade material where moisture conditions may be unfavorable.

INCREASED CONSOLIDATION PRODUCED INCREASED STABILITY FOR A GIVEN WATER CONTENT

The results of the tests indicating the relations between Hubbard-Field stability and degree of compaction for representative soil materials at constant water content are shown in figure 12. The materials used were from track 1, but the curves are typical of those that would be obtained with soils in general. Each curve in this figure represents a series of tests at a selected water content which is constant for that series. Compaction procedure was varied in each series to produce a range of densities, and a corresponding range of stabilities resulted. On all four curves, points 1, 2, 3, and 4 represent test results on samples that were molded without pretamping at pressures of 1,000, 2,000, 3,000, and 4,000 pounds per square inch, respectively. Points 5, 6, 7, and 8 represent data on samples that were pretamped and then compressed at pressures of 3,000, 4,000, 5,000, and 6,000 pounds per square inch, respectively.

The specimens containing 4 percent of water showed progressive gains in both density and stability, corresponding to greater compaction, throughout the series. Those containing 8 percent of water also showed gains in density and stability throughout the series, but these gains were much smaller for the load increments between points 5 and 8 than for the lower loads, indicating that the maximum possible density was being approached. In the series having 10 percent of water, gains were observed up to point 6, although the gain in density between points 5 and 6 was so small as to indicate that little or no further increases could be obtained without squeezing out some of the water. In fact a slight amount of water was squeezed out of specimens 3, 4, 5, and 6 during molding, reducing their actual water contents to 9.4, 9.4, 9.4, and 9.3 percent, respectively. With a water content of 9.3 percent by weight or 19.7 percent by volume and a solid mineral volume content of 79.1 percent, only 1.2 percent of the compacted volume of specimen 6 consisted of air-filled voids.



1, 2, 3 AND 4 - SPECIMENS NOT TAMPED BEFORE COMPRESSING.
5, 6, 7 AND 8 - SPECIMENS PRETAMPED TO OBTAIN HIGH DENSITY.

FIGURE 12.—RELATIONS BETWEEN HUBBARD-FIELD STABILITY AND DENSITY FOR SAND-CLAY MIXTURES.

TABLE 11.—*Change in Hubbard-Field stability accompanying change in moisture content in molded specimens*

SERIES 1

Section number	Hubbard-Field stability: Maximum	Hubbard-Field stability: At maximum density	Hubbard-Field stability: Difference	Water content: At maximum stability	Water content: At maximum density	Water content: Difference	Change in stability for a 1-percent change in water content	Behavior of test section under traffic
	Pounds	*Pounds*	*Pounds*	*Percent*	*Percent*	*Percent*	*Pounds*	
1	[1] 1,500	[1] 1,450	50	1.6	9.4	7.8	6	Remained in excellent condition throughout both compaction and test periods.
2	3,760	3,025	735	4.1	7.6	3.5	210	Unstable during compaction but excellent throughout test period.
3	3,790	3,195	595	4.6	7.2	2.6	229	Fairly stable during compaction. Generally satisfactory throughout test period.
4	4,630	2,790	1,840	3.9	6.8	2.9	634	Unstable during compaction; failed under test traffic.
5	4,520	1,975	2,545	4.2	7.4	3.2	795	Stable during compaction but failed under test traffic.

SERIES 2

Section number	Hubbard-Field stability: Maximum	Hubbard-Field stability: At maximum density	Hubbard-Field stability: Difference	Water content: At maximum stability	Water content: At maximum density	Water content: Difference	Change in stability for a 1-percent change in water content	Behavior of test section under traffic
1	3,600	[1] 1,800	1,800	8.1	9.1	1.0	1,800	Stable during compaction but failed under test traffic.
2	2,990	[1] 450	2,540	7.0	10.0	3.0	847	Do.
3	2,230	[1] 1,850	380	7.4	9.5	2.1	181	Unstable during compaction but excellent throughout test period.
4	3,530	[1] 2,100	1,430	7.2	8.5	1.3	1,100	Stable during compaction but failed under test traffic.
5	1,910	[1] 1,750	160	6.1	9.0	2.9	55	Unstable during compaction but excellent throughout test period.
6	2,800	[1] 2,200	600	5.0	8.4	3.4	176	Unstable during compaction but generally satisfactory during test period.

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[1] Interpolated from stability curves, figures 10 and 11.

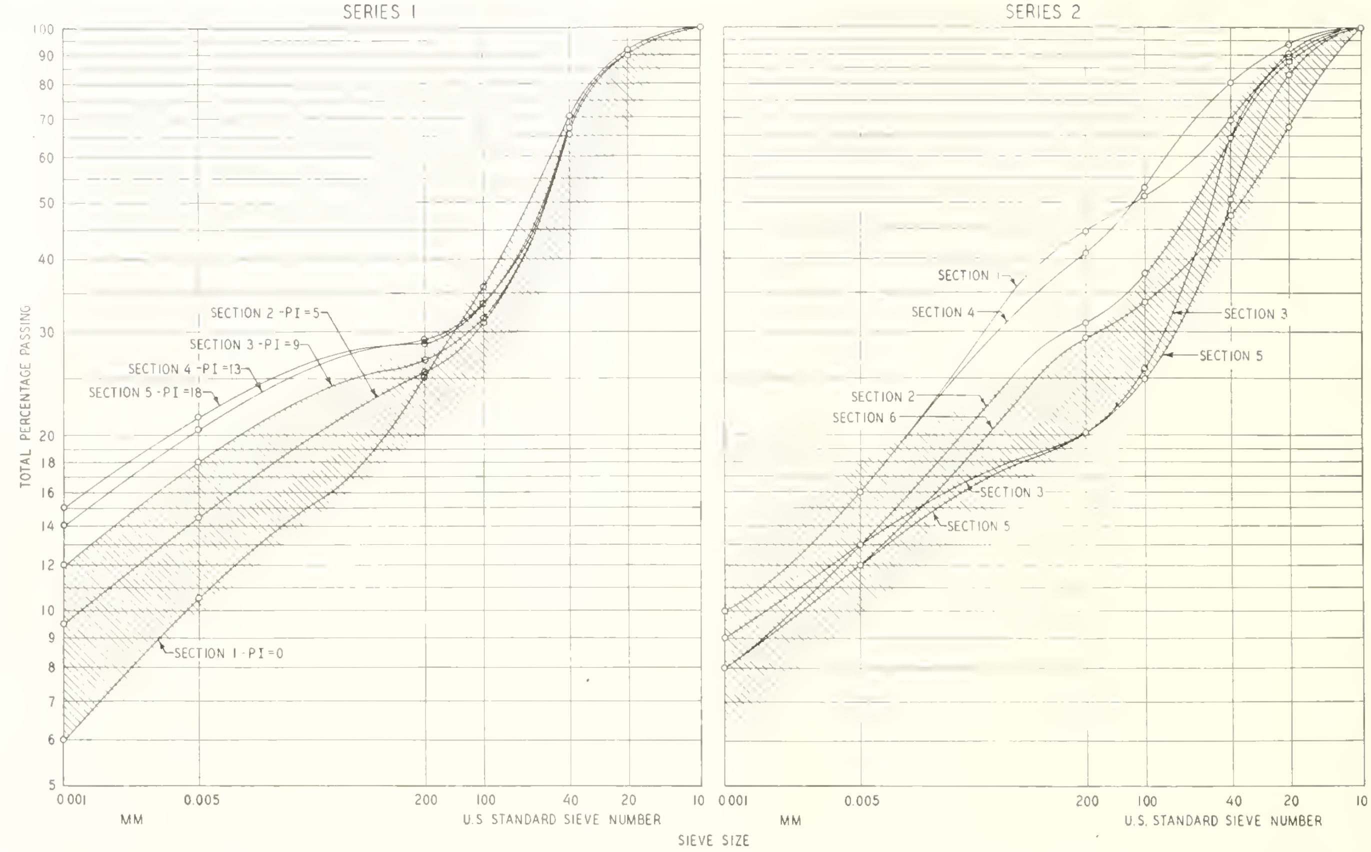

FIGURE 13.—GRADINGS OF MATERIALS IN SERIES 1 AND 2. SHADED AREAS INDICATE ZONE WITHIN WHICH ALL THE SATISFACTORY MATERIALS ARE INCLUDED.

Although, without exception, increased consolidation produced increased stability for a given water content, indicating the importance of compaction, figure 12 shows that when the maximum density was reached on a very wet mixture, the stability was still far lower than that of a fully compacted mixture having less water. It is very difficult to consolidate dry or only slightly damp materials to their maximum density by any practical field method, while the presence of an appreciable amount of moisture definitely aids consolidation. The practical solution of the problem of consolidation during construction has been found to consist of adding the amount of water that tests indicate will produce the maximum compaction for the type of equipment employed. Where the compaction is to be obtained gradually as by the action of traffic, a quantity of water somewhat in excess of the Proctor optimum moisture content is desirable since under the combined drying and compacting action the maximum density is attained.

SUMMARY

The circular track tests resulted in a quite definite classification of the 11 materials studied as to their ability to withstand the disruptive action of traffic under unfavorable moisture conditions. Moisture conditions as severe as those set up for these tests are usually avoided in good highway building practice by providing surface drainage and, where needed, subsoil drainage. However, many instances occur, especially in connection with partial or low-cost improvement of soil roads where ideal drainage cannot be provided but where improvement of the plastic properties and grading characteristics of the soil are feasible.

The tests indicated that, to guard against unsatisfactory behavior under unfavorable moisture conditions, it is necessary to control both the plastic properties and the grading of the soil. The importance of thorough compaction, even of ideal base-course materials, prior to the application of a surfacing course cannot be over-emphasized.

In series 1, where the plasticity was varied by varying the ratio of silt to red clay, the gradings were essentially constant for the five materials for all grain sizes above the No. 200 sieve but varied with the plasticity index below the No. 200 sieve, as shown at the left in figure 13. A rather wide variation was obtained in the gradings in series 2 throughout the range of particle sizes as shown at the right in figure 13. In this series the variation in plasticity index was held to a minimum, the lowest being 4 and the highest 6.

The failure of sections 4 and 5 of series 1 was caused primarily by their susceptibility to softening and becoming plastic when wet, which is a distinguishing characteristic of materials having high plasticity indexes. Section 3, having a plasticity index of 9, was classified as a border-line material, although its behavior considering the severity of the test, was quite good. The other two materials, having plasticity indexes of 5 and 0, respectively, gave excellent service.

The failure of sections 1, 2, and 4 of series 2 can be attributed to no other factor but grading. The shaded areas in figure 13 were drawn to include the gradings of all the satisfactory materials and are partially bounded by the grading curves of the two borderline materials, section 3 of series 1, and section 6 of

(Continued on page 189)

REFLECTOR BUTTONS INSTALLED ON MICHIGAN HIGHWAY

By MURRAY D. VAN WAGONER, State Highway Commissioner, Michigan State Highway Department

HIGHWAY engineers have long sought means of eliminating or counteracting the effect of darkness in augmenting ordinary traffic hazards. The recent installation of special highway markers on U. S. Highway No. 16 between Lansing and Detroit, Mich., was made in an effort to counterbalance certain of the handicaps which darkness imposes on the safe use of the highway.

That darkness is a prime factor in causing highway accidents has long been recognized. Studies of traffic volumes and accident occurrence on Michigan highways have shown that there is close correlation between the two. However, when nighttime accident and traffic figures were isolated and examined, additional facts were revealed. Sixty percent of all fatalities on trunkline highways were found to occur during the hours of darkness when these highways carried only 20 to 30 percent of their total 24-hour traffic.

The reasons for this divergence are not hard to find, though they are somewhat difficult to define. In both daylight and darkness the motorist has very similar problems of physical arrangement and movement to contend with. Although the number of vehicles with which he must share the roadway decreases, the dimensions, alinement, and surroundings of the roadway itself do not change when daylight fades. The motorist's perception of the roadway, however, is radically limited and sometimes distorted at night.

In the daytime the normal driver's vision extends far ahead and to a considerable distance on either side of the road. Some time before he traverses a section of road the driver can clearly see inherent hazards such as those involved in: (1) Traffic and pedestrians on the road; (2) road alinement and grades; (3) width and condition of road surface and shoulders; (4) roadside developments as they divert attention or obscure vision of the road ahead; and (5) roads and driveways, from which traffic can enter, cross, or leave the highway.

Although exposure to accidents is greater during the daytime concentrations of traffic, the driver can perceive the number, kind, speed, and direction of all vehicles well in advance of passing them. By being forewarned of these elements of his constantly changing driving problem, because the elements are all clearly and coincidentally visible, he ordinarily will have plenty of time to act properly to prevent serious mishaps.

Darkness blanks out practically all of these elements from the driver's sight, but not from his memory. His view of the road is limited to the short section ahead illuminated by his headlights. The rest of the picture is made up of what he knows about roads in general and what he can mentally visualize of the highway ahead through his interpretation of the lights of other cars, the dim outlines of surrounding objects, and glimpses of signs, signals, and lane markings.

At night the driver attempts to discern the location of the road beyond the range of his headlights by watching telephone lines, fences, and lights in houses, but he is conscious of the vagueness of his perceptions, particularly those involving perspective. As a result he often becomes tense and over-wary, and constantly strives to rid himself of the feeling that he is driving into a darkened tunnel or that the road ends or turns abruptly just beyond his range of sight. When facing the glare of approaching headlights, he becomes uncertain of his car's position on the roadway and instinctively draws away from the pavement's right edge, often to the extent of encroaching on the lane of opposing traffic.

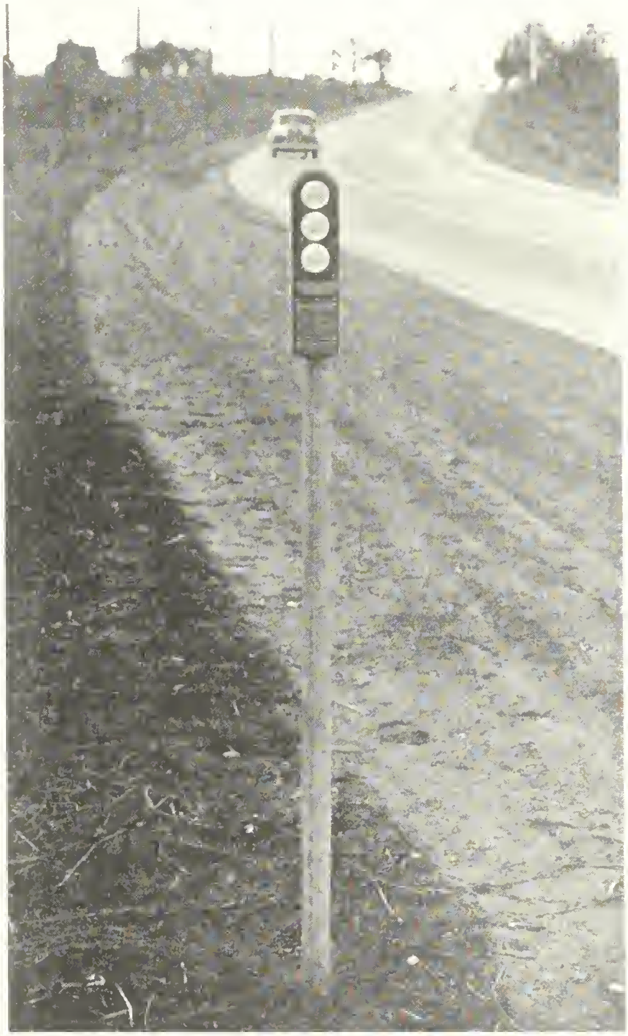

Figure 1.—A Reflector Unit as Installed on U S 16 Between Lansing and Detroit, Michigan.

The obvious way to eliminate the dangers that darkness adds to highway hazards is to duplicate, as far as practicable, daytime visibility through the use of artificial light. Attempts to solve this problem with lights on the car itself have thus far proven but partially successful. Floodlight illumination of the highway is entirely feasible from an engineering angle, and such installations have been made on a few heavily traveled highways and bridges. However, the costs of construct-

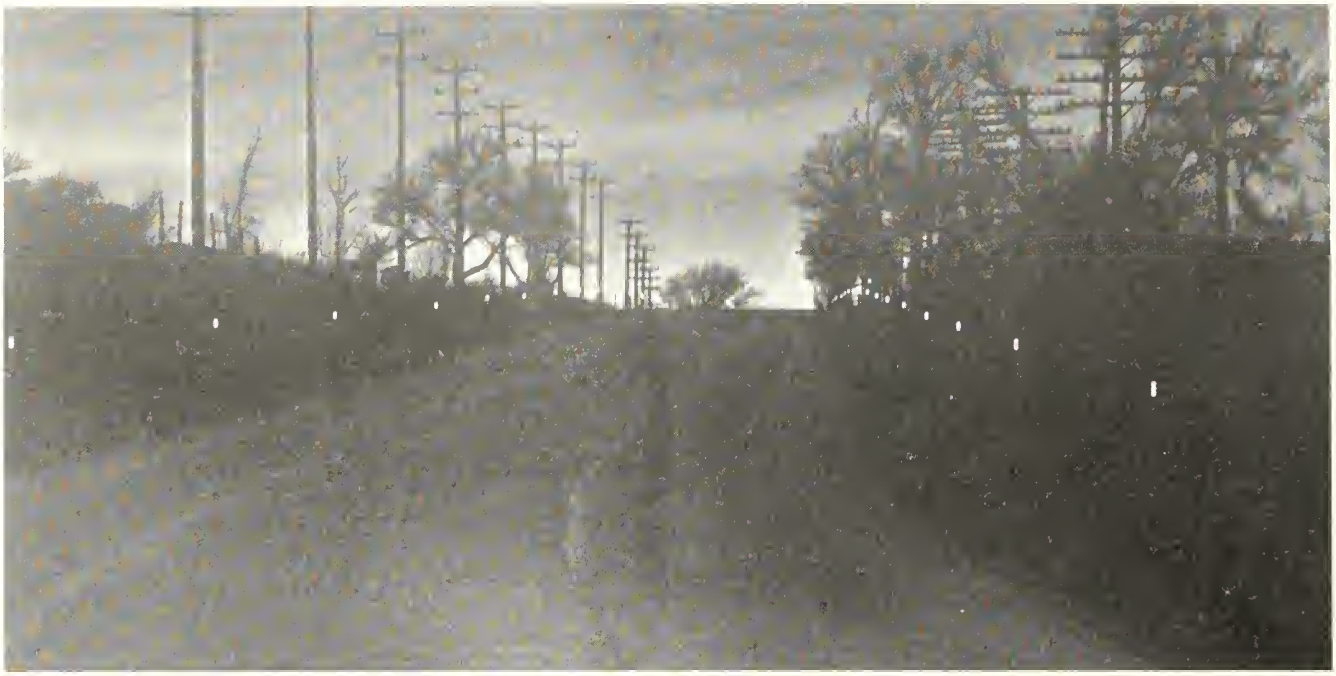

Figure 2.—Reflectors Along the Highway Illuminated by the Headlights of Automobiles at Dusk.

ing and operating highway lighting installations have been so great as to prevent the lighting of any considerable mileage of highways.

The dangers of night driving have, in the past, been somewhat mitigated by the erection of signs, the letters of which are traced by reflector buttons, to warn the driver of hazards such as narrow bridges, sharp curves, etc. The use of such buttons to make warning signs visible at night has been standard practice in Michigan and other States for many years.

Recognition of the fact that something more than the warning of specific dangers was needed to protect night traffic led to the erection of special markers to outline the highway. These markers represent the nearest approach to highway illumination without the use of expensive direct lighting that has yet been attained. Although the markers give somewhat the impression of lights spaced at regular intervals along the road, they do not illuminate the road surface to any important extent. Their principal function is to delineate the highway clearly for a distance ahead of the car considerably greater than that illuminated by the headlights.

Each marker consists of 3 reflector disks set in a vertical line in a metal holder mounted on a metal post. (See fig. 1.) The disks are 1⅝ inches in diameter, and are spaced 1⅞ inches between centers in the holder. The disks, molded from a crystal-clear synthetic resin that is nonshatterable, are lenses with many facets each having high reflecting properties.

Considerable study was given to the proper arrangement of the markers to obtain the greatest safe-driving benefits for motorists. It was found that the proper mounting height was 3 feet above the road surface. Uniformity in longitudinal spacing and offset distance from the pavement edge were essential for accurate delineation of the roadway. Consequently, the units were spaced at intervals of 100 feet, and were offset 8 feet from the pavement edge in rural areas and 4 feet in urban areas where the road surface was bordered by a curb. On rural sections having extra-wide shoulders, the offset distance was increased to 10 feet.

At places where, by using this regular spacing, units would have been placed in side roads or driveways, the unit was either moved not more than 10 feet or was omitted. Units were omitted at places where they would have interfered with traffic entering or leaving business places having broad entrance driveways. No contraction of the offset distance was permitted because of the existence of any hazard on the road shoulder between the pavement and the line of markers.

The holders, which were mounted so that the plane of the reflectors was at an angle of 90° with the centerline of the road, are of two types: Monodirectional and bidirectional. In the monodirectional holder the 3 disks are backed by metal, so that they are visible from one side only. They were used on sections where the opposing lines of traffic were separated by a central dividing strip. In the bidirectional unit each disk is backed by another disk visible from the other side. These units were used on sections carrying 2-way traffic.

The holders are stamped from 12-gage sheet metal, and are coated with yellow enamel. They can be installed on the posts without tools, and can be removed by releasing a spring lock at the bottom. The posts consist of 1¼- by 1¼- by 9/64-inch galvanized standard angle sections 6 feet long.

The location of each unit was marked by a stake placed by a survey party. Posts were driven by 40-pound pipe slip hammers developed especially for this work. A special wrench was developed for holding the posts while they were being driven.

Reflectors were installed on 61.6 miles of the highway between Lansing and Detroit, Mich. On 4.8 miles of this distance the roadways for the opposing streams of traffic were separated by a central dividing strip, and two lines of monodirectional markers were set along each roadway on this section.

The markers were installed by the Michigan State

Highway Department. The total cost of the work was $23,000, which is at the rate of $346.63 per mile, or $3.34 per unit. The cost of the labor involved in locating and installing each marker was $0.44.

Because of rigid adherence to the specified spacing and offset distance, the installation provides a simple but effective aid in night driving (figs. 2 and 3). As drivers become oriented to driving by the markers it is expected that many of the previous hazards to night driving will be eliminated.

The principal benefits derived from installing the markers are enumerated as follows:

1. The outline of the roadway is clearly visible for a considerable distance ahead, often as much as a mile where alinement and grades are favorable.

2. The presence of the light points along both sides of the roadway helps to minimize the blinding effect of undimmed headlights on approaching vehicles. It is believed that the installation may even encourage the practice of driving with dimmed lights.

3. Hazards that are present between the vehicle and the markers are revealed by the blanking out of one or more markers. Vehicles parked on the road shoulder or pavement, as well as moving vehicles without rear lights, and even pedestrians, are detected in this manner.

4. Gaps that occur because markers were omitted at road intersections and entrances to business places make the existence of these potential danger spots conspicuous.

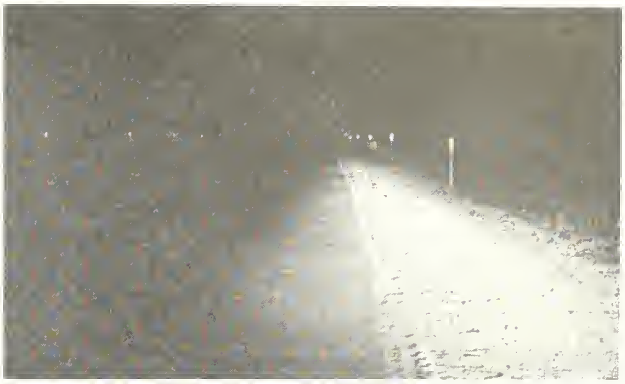

Figure 3.—Reflectors Along the Highway Illuminated by the Headlights of Automobiles After Dark.

5. Because the disks show as points of light that are fixed and uniformly spaced, they enable the driver to obtain a much more accurate sense of perspective and judgment of distance than do the lights of other cars.

6. A final advantage of strictly adhering to a uniform spacing, offset, and mounting height in this installation, and in not using the units as danger markers, is that a way is left open for possible future development along logical lines. The present installation merely outlines the roadway by reflecting white light. It is possible to use units that will reflect colored light to indicate specific details of the roadway.

(Continued from page 186)

series 2. Parts of the grading curves for all five definitely unsatisfactory materials fall outside the shaded area on the left or fine side. The plasticities of the materials in series 1 are in reality related to grading because the material that causes plasticity is distinguished primarily by its extreme fineness and, as indicated in figure 13, the plasticities of these five materials are functions of the amount of very fine aggregate included in them.

Unsatisfactory or doubtful behavior of the materials in the tests is thus clearly associated with a swing of the grading curve to the left or fine side. Apparently, such a swing adversely affects the stability of the material whether it occurs immediately below the No. 10 sieve or wholly below the No. 200 sieve.

The data indicate that satisfactory control of stability would be obtained by maintaining limits of fineness essentially as shown by the upper border of the shaded areas in figure 13. The setting up of a maximum plasticity index requirement of 6, corresponding to the highest value used in series 2 and only 1 point higher than that of the most plastic material giving wholly satisfactory service in series 1, also appears to be a necessary safeguard in specifying sand-clay materials for base-course construction.

Experience and tests have shown that poorly graded materials are less stable than well graded ones and, in order to insure a reasonable gradation from coarse to fine, minimum limits for each size fraction are believed to be essential.

Both laboratory and field experience with bituminous surfacing mixtures and soil-bound bases have repeatedly demonstrated that when the aggregate content larger than the No. 10 sieve does not exceed about 35 percent, the material retains essentially the characteristics of the mortar or fraction finer than No. 10. Although the tests described in this report were made on materials all of which passed the No. 10 sieve, the results are believed applicable to materials containing some coarser material as long as the amount of such material does not exceed 35 percent.

The results of this investigation, as well as those of a later one on coarse-graded materials, were made available to the Committee on Materials of the American Association of State Highway Officials at the time it was considering requirements for soil and gravel base courses and were made the basis for the specification given in table 12.

Table 12.—*Recommended specification for sand-clay type base course materials*

	Minimum	Maximum
Percentage of material passing No. 10 sieve	65	100
Percentage of material finer than No. 10 sieve passing		
No. 20 sieve	55	90
No. 40 sieve	35	70
No. 200 sieve	8	25
Percentage of material finer than No. 10 sieve passing No. 200 sieve (dust ratio)		50
Liquid limit (material finer than No. 40 sieve)		25
Plasticity index (material finer than No. 40 sieve)		6

CONCLUSIONS

The results of the tests justify the following conclusions:

1. Control of grading, particularly by limiting the amount of material passing the No. 40 and No. 200 sieves, is essential to insure satisfactory stability.

2. Control of grading in the sizes smaller than the No. 200 sieve is aided by establishing and maintaining a maximum limit for the plasticity index.

3. The tests demonstrated the importance of the plasticity index as a quality control. They also indicated that a plasticity index very much in excess of 6

cannot be permitted without danger of impairing the serviceability of the materials under unfavorable moisture conditions.

4. The use of an excessive amount of fine mineral dust, even though it may be relatively free of colloidal particles and therefore not productive of high plasticity, seems to retard or prevent effective compaction. This was evidenced in the tests of series 2, in which the materials having high dust contents and low plasticity indexes failed to compact well under traffic and failed soon after the water was introduced in the traffic test.

5. Thorough compaction is essential if satisfactory stability is to be obtained, particularly with base-course materials having plastic properties. Hubbard-Field stability tests on specimens having the same moisture content but different degrees of compaction showed that stability increased consistently as the amount of consolidation increased. Sections 1, 2, and 4 of series 2 failed to consolidate under traffic to densities comparable to their maximum practical densities as determined by vibratory compaction tests, and this characteristic of noncompactibility is believed to have contributed largely to their failure in the service test.

6. The noncompactibility of the unsatisfactory sections in series 2 was caused by their high dust content and is quite different in its results from the noncompactibility of section 1, series 1, which because of its harshness never became particularly dense but showed good serviceability.

7. Early difficulties encountered in compacting soils having acceptable gradings and plasticity indexes need not be taken as an indication of poor quality since in these tests such soils, without exception, gave satisfactory service when compaction in conjunction with drying was continued until essentially maximum practical density was obtained.

8. Compaction of the base courses having plastic properties should be completed to essentially maximum practical density before surfacing courses are applied because movements in the base course that are common during compaction of good materials will cause damage to prematurely constructed surfacing courses.

9. In the Hubbard-Field stability tests increases in moisture content were invariably accompanied by decreases in stability except at the very low water contents where the materials could not be properly compacted for the stability test. The rate of stability change per percent of change in moisture content, over the range of moisture contents where stability was falling while density was increasing, appears to be highly significant as a means of predicting service behavior. (See figures 10 and 11, and table 11.)

PUBLICATIONS ON BRIDGE FLOOR DESIGN AVAILABLE

Two publications that present solutions to problems of bridge floor design are now available. The solutions are applicable to various arrangements of slab and rigid or flexible supports which in the past have been susceptible of only the roughest rule-of-thumb design.

These publications, both University of Illinois Bulletins, are: No. 303, "Solutions for Certain Rectangular Slabs Continuous Over Flexible Supports," and No. 304, "A Distribution Procedure for the Analysis of Slabs Continuous Over Flexible Beams." The bulletins are the result of a cooperative investigation by the Engineering Experiment Station of the University of Illinois, the United States Bureau of Public Roads, and the Illinois Division of Highways.

The Bureau of Public Roads has a limited number of these bulletins for free distribution. Requests should be addressed to the Bureau of Public Roads, Department of Agriculture, Washington, D. C.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF OCTOBER 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$2,474,871	$1,143,635	68.3	$8,577,587	$4,277,875	361.6	$2,027,554	$1,012,685	59.3	$2,317,246
Arizona	1,529,586	1,211,663	87.7	868,732	624,897	30.1	402,971	225,311	14.5	1,239,784
Arkansas	826,321	821,912	65.3	2,394,901	2,391,298	151.0	916,805	915,930	57.0	1,098,294
California	4,516,963	2,403,557	97.1	9,134,644	4,853,656	166.6	2,245,024	1,201,220	55.2	1,380,789
Colorado	1,884,301	1,026,742	68.0	3,189,623	1,769,547	103.2	620,310	220,710	18.2	1,707,876
Connecticut				1,373,618	672,300	14.7	244,930	120,990	2.0	1,525,005
Delaware	318,655	159,140	4.0	799,097	395,550	19.5	199,690	99,845	.1	1,186,556
Florida	518,900	259,450	16.9	3,787,138	1,893,569	79.7	468,000	234,000	.3	2,981,695
Georgia	3,345,960	1,661,822	170.2	5,646,414	2,823,207	266.4	676,280	338,140	42.7	5,483,199
Idaho	1,516,494	901,846	157.1	1,613,753	962,216	71.0	88,216	52,796	5.4	1,096,155
Illinois	4,454,334	2,243,843	147.3	10,688,711	5,343,049	222.1	1,917,690	958,845	60.8	2,284,235
Indiana	2,340,690	1,170,345	70.4	5,008,669	2,504,335	102.5	1,803,024	893,466	36.7	2,181,226
Iowa	3,488,075	1,616,819	115.9	7,394,006	3,339,683	238.3	1,153,768	542,300	39.8	152,947
Kansas	2,771,755	1,384,791	406.7	4,964,881	2,438,750	420.4	3,346,652	1,672,326	153.9	2,797,831
Kentucky	2,714,554	1,344,535	100.3	4,751,912	2,375,956	136.2	1,434,094	717,047	43.8	2,073,578
Louisiana	814,659	407,227	28.9	11,742,342	2,265,858	39.6	1,761,038	775,592	28.6	2,272,042
Maine	2,209,580	1,100,369	51.2	1,651,161	825,580	34.6	569,260	284,629	10.0	84,849
Maryland	520,000	260,000	8.6	2,097,767	1,047,121	34.9	1,270,781	611,730	19.2	1,670,916
Massachusetts	1,562,989	781,493	7.1	2,639,271	1,319,632	15.1	1,175,626	585,355	12.6	2,119,705
Michigan	5,269,093	2,541,884	107.5	5,259,583	2,627,237	140.2	1,049,490	521,390	20.1	1,480,614
Minnesota	1,649,382	769,274	67.0	7,675,990	3,815,033	387.7	1,443,896	721,043	75.7	1,837,387
Mississippi	1,641,308	779,312	63.0	8,421,212	3,144,207	379.6	2,676,160	1,077,870	101.6	2,024,656
Missouri	2,945,444	1,459,173	98.0	3,249,950	1,591,110	75.0	4,532,557	2,030,444	184.4	3,024,930
Montana	1,763,004	991,238	82.1	495,573	278,176	12.5	252,912	142,262	8.4	4,361,587
Nebraska	2,273,484	1,136,634	268.5	6,206,076	3,093,437	460.9	3,332,814	1,032,854	202.7	1,830,681
Nevada	1,210,686	1,048,938	163.1	791,087	679,651	21.9	217,950	189,001	5.0	1,176,650
New Hampshire	817,799	405,206	21.1	501,386	249,733	3.8	93,802	46,900	1.4	1,141,725
New Jersey	242,315	121,100	1.9	2,786,016	1,390,128	19.8	370,230	184,490	3.3	2,505,753
New Mexico	1,642,148	1,001,044	187.5	1,805,471	1,201,322	100.6	841,369	513,146	47.0	340,604
New York	5,628,509	2,757,475	123.6	13,839,330	6,846,742	223.8	4,717,370	2,174,495	73.2	1,494,619
North Carolina	4,610,957	2,186,400	180.3	5,554,799	2,771,532	322.1	1,446,660	702,530	88.3	1,784,819
North Dakota	2,039,728	1,928,527	173.6	1,739,781	1,530,474	138.0	83,622	44,787	7.4	3,568,606
Ohio	4,282,540	2,114,689	53.1	9,460,472	4,709,270	87.0	2,811,060	1,395,163	36.7	5,700,481
Oklahoma	3,580,717	1,890,953	150.4	4,464,673	2;323,247	138.2	1,172,660	622,764	38.9	2,601,051
Oregon	2,222,570	1,296,270	89.2	1,251,206	755,997	22.6	636,823	388,490	85.1	1,786,618
Pennsylvania	5,560,911	2,759,048	95.2	8,246,282	4,115,964	96.6	2,771,445	1,369,252	26.5	2,691,996
Rhode Island	559,790	279,895	7.7	978,112	489,056	12.2	73,900	36,950	.6	1,028,924
South Carolina	3,244,245	1,431,448	191.0	2,975,292	1,325,376	95.0	1,722,221	788,500	49.0	1,232,875
South Dakota	1,176,622	666,065	143.0	4,466,176	2,469,840	399.0	1,235,890	682,900	131.2	2,540,329
Tennessee	2,586,220	1,293,110	94.1	4,036,354	2,018,177	93.0	1,267,040	633,020	30.6	4,046,640
Texas	8,408,382	4,170,715	552.8	11,055,979	5,323,067	531.5	3,527,337	1,696,440	159.8	5,544,952
Utah	777,266	541,930	104.2	1,915,150	1,364,145	62.2	296,595	208,615	13.7	703,283
Vermont	1,010,022	459,608	28.4	881,090	415,883	23.4	309,470	154,545	5.4	158,292
Virginia	3,792,051	1,894,195	126.0	4,306,764	2,151,841	145.1	1,158,260	576,780	29.9	563,911
Washington	2,730,290	1,435,225	59.0	2,852,944	1,495,259	42.2	934,709	491,500	11.0	686,362
West Virginia	796,020	572,573	31.0	2,078,993	1,272,718	57.2	840,440	414,565	19.8	2,026,161
Wisconsin	3,204,269	1,554,071	96.5	7,171,759	3,350,450	210.5	472,173	217,400	19.0	1,458,413
Wyoming	1,845,727	1,136,088	213.8	844,002	515,981	92.8	343,970	212,540	51.0	517,704
District of Columbia										
Hawaii	513,335	256,538	9.9	1,126,605	554,362	17.2	28,450	11,865	.3	1,202,411
Puerto Rico				1,128,739	562,390	20.5	63,720	31,660	4.2	640,325
TOTALS	115,833,521	60,777,815	5,253.5	215,891,073	106,555,884	6,939.6	63,046,708	30,777,078	2,191.3	97,357,287

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF OCTOBER 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 73,700	$ 36,850	10.7	$ 488,300	$ 243,850	27.1	$ 344,805	$ 170,550	11.8	$ 601,722
Arizona	281,361	187,459	18.6	111,297	70,491	6.8	86,796	51,357	10.7	347,175
Arkansas				19,426	12,863		195,706	195,541	21.5	655,704
California	484,307	279,472	45.9	1,231,089	668,519	51.6	772,772	392,914	40.1	506,321
Colorado	439,466	243,558	31.6	581,370	322,425	30.1	156,140	65,010	7.3	290,575
Connecticut				79,344	39,512	1.3	38,340	19,135	.3	259,631
Delaware				59,680	29,840	10.9	44,024	22,012	6.0	195,023
Florida				20,122	10,061		293,782	146,850	4.9	517,941
Georgia	197,440	96,612	23.8	483,113	244,059	61.2	292,280	146,140	44.0	771,911
Idaho	240,507	110,501	40.2	268,483	117,491	17.1				246,379
Illinois	775,700	387,850	69.7	1,569,392	730,696	107.6	764,900	373,950	47.1	534,419
Indiana	370,100	144,250	42.9	818,500	391,450	77.9	798,948	384,152	68.9	322,586
Iowa										1,298,449
Kansas	43,670	21,835	.5	95,410	47,705	13.3	236,972	118,485	42.7	1,130,946
Kentucky	486,158	142,751	64.0	829,560	233,659	55.6	868,143	255,398	77.6	182,618
Louisiana				267,530	119,085	21.9	801,332	335,820	67.1	269,231
Maine	240,400	120,200	15.8	378,226	183,996	19.9	18,700	9,350	1.2	15,027
Maryland				45,164	15,787	5.8	237,800	95,400	12.7	301,289
Massachusetts				78,010	39,005	1.4	237,740	118,080	4.9	489,315
Michigan	41,961	19,481	7.1	796,744	398,372	61.2	290,250	145,125	10.3	972,561
Minnesota	217,978	101,203	30.8	484,284	237,735	53.3	262,924	131,462	10.4	909,180
Mississippi				299,000	149,500	23.8				739,427
Missouri	255,371	125,865	35.2	197,480	98,350	14.4	573,710	253,365	103.2	588,092
Montana				13,983	7,865					1,027,170
Nebraska	299,900	149,950	47.4	406,738	203,369	67.2	516,788	248,693	90.0	394,745
Nevada	344,302	295,660	50.8	102,333	86,034	24.7	48,020	41,642	3.5	71,016
New Hampshire	97,426	48,277	1.8	151,155	75,181	4.3	81,281	39,806	2.4	83,611
New Jersey				274,120	135,215	5.0	43,780	17,500	.1	520,058
New Mexico	404,779	246,873	17.6	333,076	183,006	23.4	470,128	234,453	26.7	142,440
New York	1,673,234	833,319	118.0	2,107,000	1,053,500	122.1	452,800	213,900	25.8	261,776
North Carolina	302,920	151,460	36.3	980,204	486,630	84.7	208,720	96,490	18.0	347,275
North Dakota	33,360	18,135	5.5	119,670	64,092	29.3	115,886	62,065	8.5	642,960
Ohio	156,560	78,280	3.8	27,840	13,920		164,600	81,750	10.2	1,657,691
Oklahoma	12,200	6,491		396,738	209,979	34.3	449,030	215,734	29.9	752,711
Oregon	376,674	228,080	53.5	80,975	49,452	2.5	55,431	24,900	7.6	452,701
Pennsylvania	1,108,373	534,838	84.9	1,485,615	732,319	80.6	1,110,998	517,049	58.8	332,478
Rhode Island	66,840	33,420	3.5	121,188	60,594	4.0	131,140	65,570	1.7	28,307
South Carolina	155,841	69,232	16.6	811,345	345,230	93.4	300,420	112,900	28.2	151,318
South Dakota				11,300	6,250					816,436
Tennessee	78,900	39,450	2.2	631,306	242,753	34.3	159,480	79,740	6.0	697,547
Texas	1,080,001	505,260	177.8	2,053,183	897,470	220.0	1,000,701	434,555	124.6	1,287,484
Utah	375,496	205,870	38.8	226,449	120,125	15.9	75,170	37,360	8.7	187,015
Vermont	192,650	84,150	11.6	138,306	69,153	6.2	43,300	20,500	.5	35,793
Virginia	333,947	164,758	43.4	824,230	362,175	54.3	99,190	49,595	8.5	315,471
Washington	400,253	210,000	43.0	257,753	135,578	22.1	413,793	202,600	26.0	182,378
West Virginia	124,300	62,150	9.5	119,750	59,875	11.9	108,950	54,475	6.2	373,174
Wisconsin	141,919	62,885	4.6	989,096	481,460	47.8	109,051	54,000	2.1	613,883
Wyoming	389,390	240,600	53.9	302,002	186,599	12.6	187,238	115,691	14.8	42,303
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico				272,250	135,650	15.2				111,225
TOTALS	12,297,884	6,287,025	1,261.3	22,500,984	10,836,050	1,680.4	13,666,959	6,451,064	1,101.5	23,893,238

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . . The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP . . Roadside Improvement. 10 cents.

No. 272MP . . Construction of Private Driveways. 10 cents.

No. 279MP . . Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

DEPARTMENT BULLETINS

No. 1279D . . Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D . . Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . . Highway Bridge Surveys. 20 cents.

No. 265T. . . Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . . Bibliography on Highway Safety.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF OCTOBER 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$220,910	$220,910	5		4	$544,948	$543,924	6			$428,505	$427,600	5		3	$793,085
Arizona						9,452	9,452									620,761
Arkansas	46,565	45,628	3			423,519	423,299	7			124,319	124,039	4			1,165,803
California	366,760	366,760		2		1,266,665	1,266,090	7	2		196,046	196,046	2			1,871,313
Colorado	30,465	27,839	1		1	61,728	61,728	1			38,842	35,082			16	1,163,663
Connecticut						18,930	12,665									831,825
Delaware	11,500	11,500			3						65,770	65,770			20	416,480
Florida						196,216	196,216	2			207,100	207,100	1			1,002,481
Georgia						45,786	45,786	1			338,250	338,250	5		1	2,033,351
Idaho	87,825	87,652	1			177,264	177,264	3			173,299	159,270	4			398,684
Illinois	186,500	186,500	1		1	1,544,675	1,544,675	5	2		896,230	896,230	9		60	2,577,238
Indiana	408,486	322,500	2	3		462,000	435,100	3	2		689,780	689,780	1	1	89	966,769
Iowa	640,258	606,297	5		11	570,770	538,800	7	1	6	36,420	33,900		2	4	1,388,564
Kansas	456,074	456,074	8		4	383,515	383,515	5			675,801	675,801	8		7	1,059,118
Kentucky	145,000	145,000	1			51,100	51,100	1			584,830	584,830	7	2	31	1,033,115
Louisiana						225,285	196,478	3			248,872	248,090	9			1,132,503
Maine	48,590	48,590	2			327,315	327,315	3	2		69,830	69,830	1		1	245,468
Maryland						64,586	64,586	1								962,247
Massachusetts						275,196	275,000	1	2		235,125	233,194		3		1,405,094
Michigan	844,383	837,972	7	1	7	689,516	689,516	7	1	38	81,290	81,290		1		1,676,407
Minnesota	40,218	40,218		1	1	770,180	770,180	3	5		5,589	5,589			1	1,629,147
Mississippi						406,900	406,900	5			308,900	308,900	2			868,351
Missouri	128,120	128,120	2	1		223,970	223,970	2			453,520	453,520	2	1		2,219,643
Montana	248,112	243,230	3			556,992	556,992	5								524,249
Nebraska	150,374	150,374	4			390,393	390,393	10			556,270	556,270	8		33	667,373
Nevada	97,187	97,187		2	2						157,121	157,121	1	1	3	204,333
New Hamoshire	61,732	61,425		1		93,487	93,428	5			4,590	4,590			2	334,307
New Jersey						210,005	204,779	2	1		130,800	130,800		1	2	1,634,678
New Mexico	109,669	109,669	4			196,671	196,671	3	1							545,151
New York	141,400	141,400		1	2	2,447,252	2,438,751	9	8		148,600	148,150		1		4,041,628
North Carolina	73,550	73,550		1		771,710	739,010	6	4		300,730	300,730	2	1	11	1,331,104
North Dakota						606,278	606,278	2	2							972,943
Ohio						95,850	95,850	2			640,700	640,700	7			3,492,618
Oklahoma						17,343	17,343				129,105	128,305			34	2,135,861
Oregon	308,391	307,742	1	2		58,634	58,634									608,955
Pennsylvania	138,043	122,837	1			161,073	161,073	1			143,528	138,621	1			5,310,004
Rhode Island						335,019	335,019	1	2							158,731
South Carolina						169,559	114,593	3	2		215,610	215,610	10		38	1,158,899
South Dakota	91,230	91,230	1		6	250,628	250,628	2	2	3	90,870	90,870	4		2	887,372
Tennessee						14,381	14,381				105,010	105,010		2	5	1,772,642
Texas	34,527	33,861	2			1,219,529	1,218,452	12	5		301,040	254,605	3			3,836,198
Utah	97,174	97,174	2		1	47,359	47,359	2			5,319	5,319			3	423,211
Vermont	202,882	197,882	6	1		29,964	29,964		1	1	31,640	31,640			11	220,950
Virginia	245,670	245,670	12	1		241,018	241,018	3	1		386,338	386,338	4	1	8	923,004
Washington	149,718	147,618	1	2	2	613,456	613,456	7	2		87,926	87,926		1	8	534,800
West Virginia	120,100	119,080	1			315,009	315,009	1			40,070	40,070	2		6	849,939
Wisconsin	162,493	162,493	2			1,246,886	1,160,026	12								1,150,990
Wyoming	30,890	30,890		1	1	135,630	135,630		1		15,960	15,960			6	494,250
District of Columbia											34,262	34,262		1		291,169
Hawaii						33,230	33,230	1			152,570	152,570	2	1		307,950
Puerto Rico						214,569	213,370	4								516,930
TOTALS	6,124,796	5,964,872	78	20	46	19,211,441	18,924,896	166	49	48	9,536,377	9,459,578	104	20	405	62,791,349

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 10 DECEMBER 1938

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the
UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS
D. M. BEACH, *Editor*

Volume 19, No. 10 December 1938

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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PRINCIPLES OF SOIL MECHANICS INVOLVED IN THE DESIGN OF RETAINING WALLS AND BRIDGE ABUTMENTS

BY THE DIVISION OF TESTS, BUREAU OF PUBLIC ROADS

Reported by **L. A. PALMER**, Associate Chemist

IN A LARGE NUMBER of earth pressure and foundation problems the stresses found by the method based on elasticity are independent of elastic constants and are classified as problems of plane strain or plane deformation. These are problems involving two dimensions, and in their solution an analysis is made of the stresses in a vertical cross section of the earth embankment or supporting soil under a foundation. The limitations of the analytical method based on the assumption of the conditions of plane strain are indicated in this paper in the case of the supporting soil under abutments, piers, and retaining walls.

The principle of superimposition of different systems of loading is applied to problems of plane strain in deriving expressions for the greatest shearing stress in the undersoil below a symmetrical fill and a bridge abutment. The greatest unit shearing stress anywhere in the undersoil below a fill having equal side slopes and the top surface parallel to the subgrade surface is approximately $0.3p$, where p is the unit pressure of fill material at the subsoil level beneath the roadway. If $0.3p$ is less than the unit cohesion c of the undersoil, the latter is safe insofar as ultimate failure is concerned. However, when $0.3p$ exceeds c of the undersoil, it does not follow that failure is inevitable. In such a case Prandtl's formula is applied,

$$q=(c \cot \phi+wb' \cot \alpha)\left[\frac{1+\sin \phi}{1-\sin \phi}e^{\pi \tan \phi}-1\right]$$

where, with respect to the undersoil,

q = supporting power,
c = unit cohesion,
w = unit weight,
ϕ = angle of internal friction,
$\alpha = 45° - \phi/2$, and
$b' = \frac{1}{2}$ the horizontal distance between the midpoints of the slopes of the symmetrical fill.

By this formula, the factor of safety against failure of the supporting soil is the ratio, $\frac{q}{p}$.

The maximum shearing stress in the soil below the base of an abutment subjected to a rotating moment producing a maximum vertical pressure p_0 at the toe is $\frac{p_0}{\pi}$. If $\frac{p_0}{\pi}$ is less than the cohesion of the undersoil, it will not fail under this stress. However, if $\frac{p_0}{\pi}$ exceeds c, it is necessary to determine the factor of safety $\frac{q}{p_0}$ against failure, obtaining q from the formula,

$$q=\frac{2c}{\tan \alpha \sin^2 \alpha}+\frac{wa}{2 \tan \alpha}\left[\frac{1}{\tan^4 \alpha}-1\right]+\frac{wd}{\tan^4 \alpha}$$

where

q = supporting power,
c = unit cohesion,
w = unit weight,
$\alpha = 45° - \phi/2$,
d = depth of surcharge, and
a = width of the base of the abutment.

The active and passive earth pressures and the earth pressure at rest, in the earth back of retaining walls, are easily determined by using the analytical method of Coulomb and the graphical method of Mohr. The conditions of plane strain are assumed in the application of both methods. Assuming a level earth surface back of the retaining wall, the expressions for the three earth pressures are

$$p_h \text{ (active)} = p_v \tan^2 (45° - \phi/2) - 2c \tan (45° - \phi/2)$$

$$p_h \text{ (at rest)} = Kp_v$$

and

$$p_h \text{ (passive)} = p_v \tan^2 (45° + \phi/2) + 2c \tan (45° + \phi/2)$$

where

ϕ = angle of internal friction of embankment material,
c = unit cohesion of embankment material,
K = coefficient of earth pressure at rest, of the embankment material,
p_h = horizontal pressure, and
p_v = vertical pressure.

The values, c, ϕ, and K are determined experimentally; p_h (active) is the smallest in magnitude of the earth pressures; p_h (passive) is the largest; and the earth pressure at rest, p_h (at rest) is intermediate in value between these two extremes. If the earth back of the wall moves outward when the wall fails by rotation about the base, the pressure is active. If movement of the wall is exceedingly small or zero, the pressure against it is earth pressure at rest. If the soil back of the wall is cohesive and of a nature such that it tends to expand or swell on wetting and to contract on drying, then, on swelling, the pressure exerted on the wall is passive earth pressure, and the wall should be designed to withstand passive earth pressure.

If the earth back of the wall is cohesionless material such as sand, the maximum pressure against the wall is earth pressure at rest. The value of K for sand has been reported as being within the limits 0.40 to 0.45.

The object of this paper and that of a former one [1] is to make available for practicing engineers the method based on elasticity as a theoretical and practical approach to the study of earth problems. This method is particularly applicable to a large number of problems falling under the heading of plane strain. In this case the stresses found by the method based on elasticity are independent of the elastic constants, and the difference in the behavior of two different earth materials under loads which are equal and identical in distribution and with the same plane strain conditions is entirely a matter of difference in the relative displacements of the two stressed materials.

The body under stress is assumed to be very long in comparison with its width, the length being in the Y direction. In this class of problems the stresses and strains are independent of one of the three rectangular coordinates, the Y coordinate in this paper. Examples of this class most frequently considered in textbooks are stresses in thick-walled tubes under internal pressure and stresses produced in a sheet of metal during rolling in which the dimension perpendicular to the direction of rolling is very large. Obviously, neither the tube nor the sheet are of "infinite length," an expression often used in purely mathematical treatments of such problems. In foundation problems, a foundation wall, a fill, an earth dam, a long retaining wall, or a bridge abutment with wing walls may be considered as examples of the plane strain problem.

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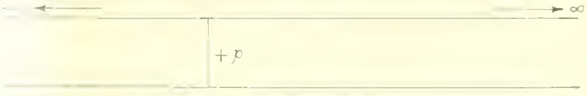
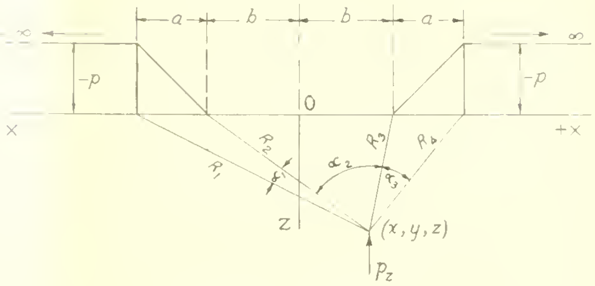
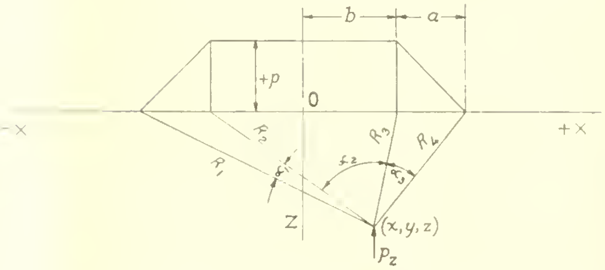
[1] The Theory of Soil Consolidation and Testing of Foundation Soils, by L. A. Palmer and E. S. Barber, PUBLIC ROADS, vol. 18, No. 1, March 1937.



(1). UNIFORM POSITIVE LOAD DIAGRAM



(2). NEGATIVE LOAD DIAGRAM



(3). POSITIVE LOAD DIAGRAM

FIGURE 1.—A UNIFORM LOAD (1), SUPERIMPOSED ON A NEGATIVE LOAD DIAGRAM FOR A CUT (2), YIELDS THE POSITIVE LOAD DIAGRAM FOR A FILL (3).

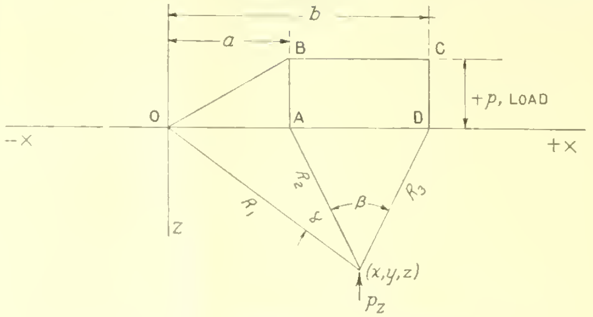
CALCULATIONS INVOLVE SUPERIMPOSITION OF VARIOUS SYSTEMS OF LOADING

In this paper formulas will be derived for stresses in the supporting earth below (a) a symmetrical fill and (b) a bridge abutment subjected to a rotating moment of such a nature that a trapezoidal load distribution is transmitted by the abutment. Recourse will be had to well known methods of the superimposition of loading systems in these derivations. Further application of the principles of plane strain will be made in the analysis of the three types of earth pressures against retaining walls: (1) Earth pressure at rest; (2) active earth pressure; and (3) passive earth pressure. The method based on plastic equilibrium will be used in deriving an expression for the bearing capacity of soil supporting a bridge abutment and finally, there will be indicated by direct computations the extent of application of the methods based on plane strain in determining earth stresses below loaded rectangular areas of different sizes. These are topics of vital interest to bridge engineers and a continuation of the analysis much beyond the scope of this paper is needed.

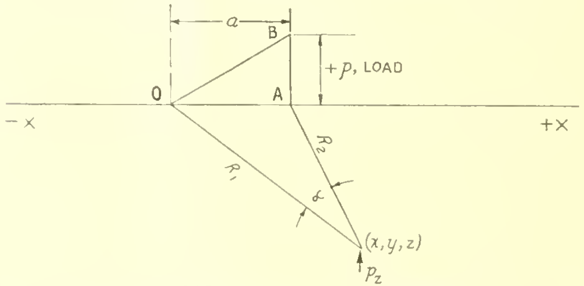
Working formulas have been developed by S. D. Carothers [2] for several different systems of loading, all of which pertain to foundation problems of the plane strain class. It is possible to derive formulas for various additional loading systems by superimposing two or more of those already considered by Carothers. Two examples of this procedure will be given:

1. A uniform load, p, is superimposed on the negative load diagram for a cut so as to yield the load diagram for a fill (see fig. 1). The loads are infinite in extent in both the X and Y directions, the Y direction being perpendicular to the plane of the figure.

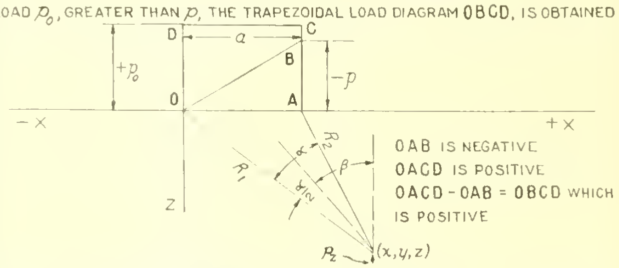
2. By first taking $b=a$ in the upper diagram, $OBCD$, of figure 2, there is obtained a system of triangular loading, OAB, over a long strip as indicated. If this triangular load is then given a negative sign, and there is then superimposed on it a strip of uniform pressure, p_0, as shown in the lower diagram, there results a trapezoidal load distribution, $OBCD$, over a long strip.



FOR $b=a$, THE LOAD DIAGRAM OBCD BECOMES OAB, THEN $R_3=R_2$ AND $\beta=0$



BY TAKING LOAD DIAGRAM OAB AS NEGATIVE AND SUPERIMPOSING ON IT A UNIFORM LOAD p_0, GREATER THAN p, THE TRAPEZOIDAL LOAD DIAGRAM OBCD, IS OBTAINED



OAB IS NEGATIVE
OACD IS POSITIVE
OACD - OAB = OBCD WHICH IS POSITIVE

FIGURE 2.—DEVELOPMENT OF DIAGRAM SHOWING TRAPEZOIDAL LOAD DISTRIBUTION OVER A LONG STRIP.

Example 1—Derivation of formulas for stresses beneath a fill.—In problems of plane strain it is usually sufficient to know six stresses: The normal stresses, p_x and p_z, acting in the directions of the OX and OZ axes, respectively; the principal stresses, p_1 and p_2, acting at right angles to each other and perpendicular to planes of zero shearing stress; the shearing stress, s_{xz}, corresponding to the normal stresses, p_x and p_z; and the maximum shearing stress, $s_{\max.}$ at any point, equal in magnitude to half the difference of the two principal stresses at that point. That is, at any point (x, y, z) in the stressed undersoil,

$$s_{\max.}=\frac{p_1-p_2}{2} \quad \text{-------------} \quad (1)$$

where p_1 is the major and p_2 the minor principal stress. Since displacements in the undersoil in the direction of the OY axis (perpendicular to the XOZ plane) are zero, it is usually unnecessary to consider the normal stress, p_y, acting in this direction. It is necessary, however, to consider the magnitude of p_y at the terminal of a long loaded strip or fill where the retaining wall or abutment is placed. In computing p_1, p_2, and $s_{\max.}$, p_y is not needed in problems of plane strain. In texts[3] dealing with the theory of elasticity, the following relations are derived:

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[2] Direct Determination of Stresses, by S. D. Carothers, Proceedings of the Royal Society of London, Series A, vol. 97, p. 110 et seq.
Elastic Equivalence of Statically Equipollent Loads, by S. D. Carothers, Proceedings of the International Mathematical Congress, vol. 2, Toronto University Press, 1924. p. 527 et seq.
Test Loads on Foundations as Affected by Scale of Tested Area, by S. D. Carothers, Engineering, London, July 4, 1924, p. 1 et seq.

$$p_1=\frac{p_x+p_z}{2}+\left[\left(\frac{p_x-p_z}{2}\right)^2+(s_{xz})^2\right]^{1/2} \text{-----} \quad (2)$$

$$p_2=\frac{p_x+p_z}{2}-\left[\left(\frac{p_x-p_z}{2}\right)^2+(s_{xz})^2\right]^{1/2} \text{-----} \quad (3)$$

$$s_{\max.}=\frac{p_1-p_2}{2}=\left[\left(\frac{p_x-p_z}{2}\right)^2+(s_{xz})^2\right]^{1/2} \text{-----} \quad (4)$$

Hence, if at any point (x, y, z) of the undersoil, the stresses, p_x, p_z, and s_{xz}, are known, values for p_1, p_2, and $s_{\max.}$ are easily derived.

ISOSHEAR LINES DETERMINED FOR MATERIAL BENEATH A FILL

With reference to the load diagram for a cut in figure 1, the vertical pressure, p_z, at a point (x, y, z) in the undersoil as derived by Carothers was found to be

$$p_z \text{ (for cut)}=\frac{p}{\pi a}[\pi a-a(\alpha_1+\alpha_2+\alpha_3)$$
$$-b(\alpha_1+\alpha_3)-x(\alpha_1-\alpha_3)] \text{----------} \quad (5)$$

Formula (5) as written is for compression. If p is given a negative sign, then p_z is tension. If a uniform compression, p, is added to this tension, i. e., if the expression on the right of formula (5) is subtracted from p, there results the formula for the vertical pressure, p_z, at the same point as produced by the load diagram for a fill, figure 3. That is,

$$p_z \text{ (for fill)}=p-\frac{p}{\pi a}[\pi a-a(\alpha_1+\alpha_2+\alpha_3)$$
$$-b(\alpha_1+\alpha_3)-x(\alpha_1-\alpha_3)]$$

or

$$p_z \text{ (for fill, fig. 3)}=\frac{p}{\pi a}[a(\alpha_1+\alpha_2+\alpha_3)$$
$$+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)] \text{----------} \quad (6)$$

where the angles, α_1, α_2, and α_3, are expressed in radians.

Similarly, by taking Carothers' formulas for p_x and s_{xz} for the cut and considering p_x and s_{xz} as produced by the uniform compression as p and zero, respectively, there is obtained

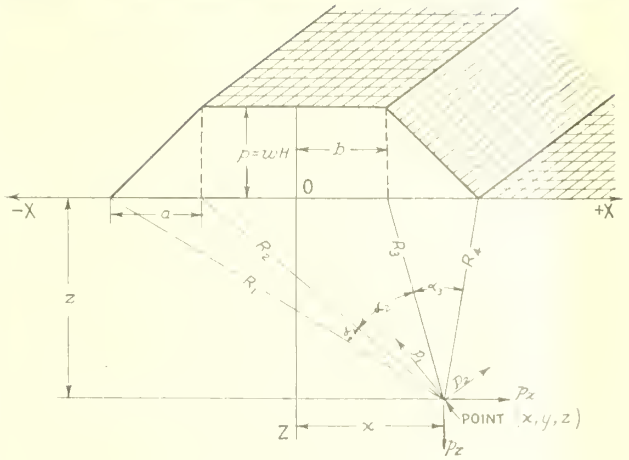
$$p_x \text{ (for fill)}=\frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)\right.$$
$$\left.+x(\alpha_1-\alpha_3)-2z\log_e\frac{R_1R_4}{R_2R_3}\right] \text{----------------} \quad (7)$$

and

$$s_{xz}=-\frac{zp}{\pi a}(\alpha_1-\alpha_3) \text{------------} \quad (8)$$

By substituting the values for p_z, p_x, and s_{xz}, as given in formulas (6), (7), and (8), in formulas (2), (3), and (4), expressions for the stresses, p_1, p_2, and $s_{\max.}$, are obtained. Thus

$$s_{\max.}=\frac{zp}{\pi a}\left[\log_e{}^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2\right]^{1/2} \text{-------} \quad (9)$$



$$p_z=\frac{p}{\pi a}[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)]$$

$$p_x=\frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)\right.$$
$$\left.-2z\log_e\frac{R_1R_4}{R_2R_3}\right]$$

$$s_{xz}=-\frac{zp}{\pi a}(\alpha_1-\alpha_3)$$

$$p_1=\frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)\right.$$
$$\left.-z\log_e\frac{R_1R_4}{R_2R_3}\right]+\frac{zp}{\pi a}\sqrt{\log_e{}^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2}$$

$$p_2=\frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)\right.$$
$$\left.-z\log_e\frac{R_1R_4}{R_2R_3}\right]-\frac{zp}{\pi a}\sqrt{\log_e{}^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2}$$

$$s_{\max.}=\frac{zp}{\pi a}\sqrt{\log_e{}^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2}$$

FIGURE 3.—LOAD DIAGRAM AND COMPUTATION OF STRESSES IN EARTH BELOW A FILL.

Along the axis, OZ, $R_1=R_4$, $R_2=R_3$, and $\alpha_1=\alpha_3$. Hence formula (9) reduces to

$$s_{\max.}=\frac{zp}{\pi a}\log_e\left[\frac{R_1}{R_2}\right]^2=\frac{2zp}{\pi a}\log_e\frac{R_1}{R_2} \text{-----} \quad (10)$$

For $a=b$, figure 3, and for points along OZ, it is found by trial that the greatest value of $s_{\max.}$ is at the point, $z=\frac{3}{2}a$, where $s_{\max.}=0.31p$ (or $0.3p$ approximately) Similarly, by taking $a=2b$, the greatest value of $s_{\max.}$ at any point on the center line, OZ, is at $z=0.96a$, where $s_{\max.}=0.30p$. It is worthy of note that these relations are independent of the height, H, of the fill. By varying the height and the slope, and keeping $\frac{a}{b}$ the same, the relation, greatest $s_{\max.}=0.3p$ at $z=\frac{3}{2}a$ for $a=b$ or at $z=0.96a$ for $a=2b$, is not changed, although the magnitude of p and therefore, of $s_{\max.}$, increases as H is increased. Here $p=wH$, where w is the average unit weight of fill material.

By drawing accurately and to scale the load diagram, the radial lines, R_1, R_2, R_3, and R_4, the vertical and

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[3] See, for example, pp. 16 to 19, inclusive, of Theory of Elasticity, by S. Timoshenko. McGraw-Hill Book Co., 1934.

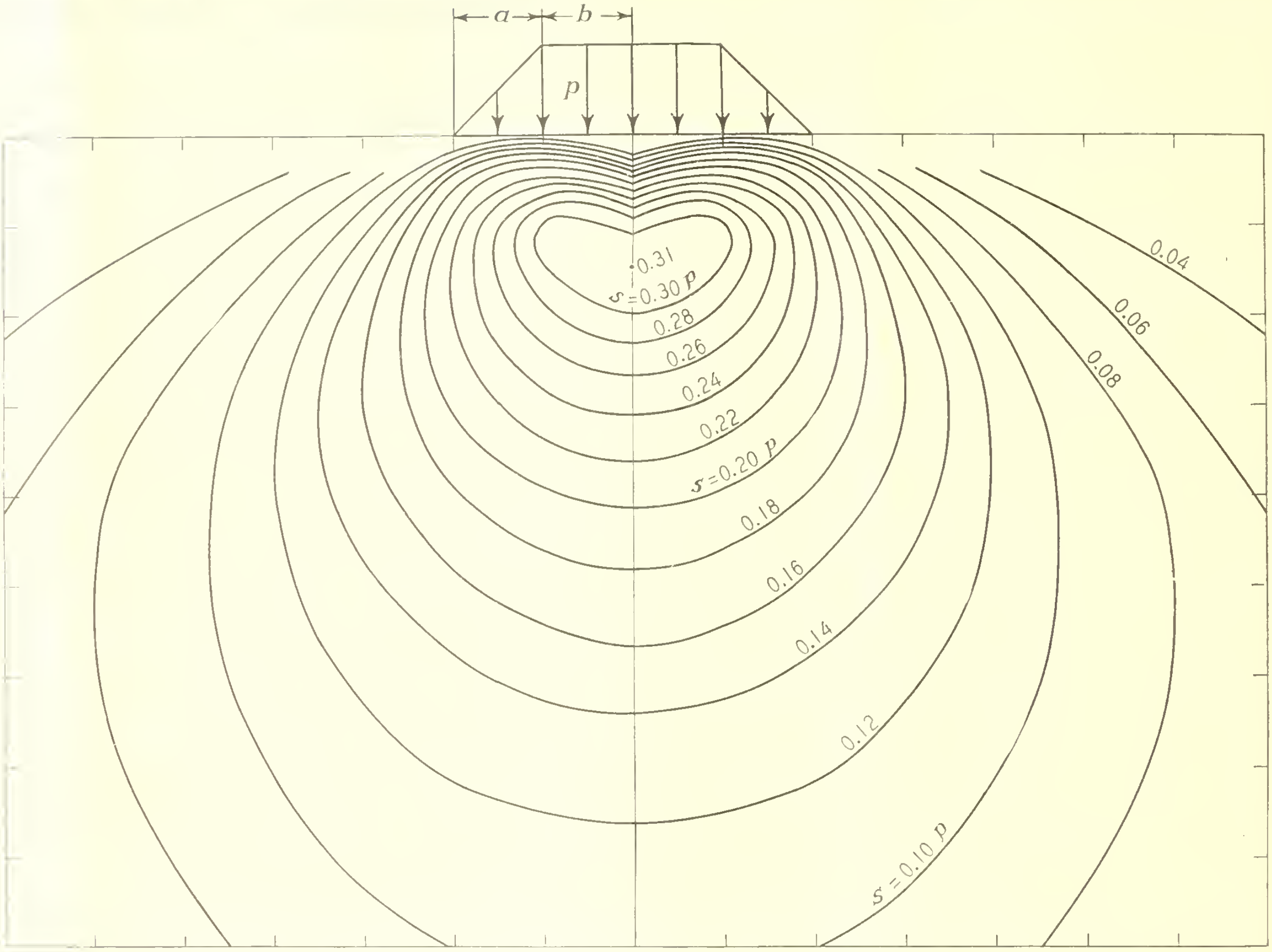

Figure 4.—Isoshear Lines Under Typical Fill, Where $a=b$.

horizontal distances z and x, respectively, and the subtended angles, α_1, α_2, and α_3, the numerical values of $s_{\max.}$ may be computed for various points in the supporting soil. All points of the same $s_{\max.}$ may be connected by a smooth curve, and a series of such curves or isoshear lines, as shown in figure 4, may be drawn.

It has been shown[4] that if the greatest value of $s_{\max.}$, namely $0.3p$, does not exceed the unit cohesion (of the undersoil), the supporting power of the undersoil is usually ample, and that if $0.3p$ exceeds the unit cohesion, a further analysis of the supporting power of the undersoil should be made.

If $0.3p$ exceeds the unit cohesion c of the undersoil, it does not follow that failure of the supporting earth is inevitable, but it is necessary in such a case to determine the supporting power q by Prandtl's formula,[5] which is

$$q=(c \cot \phi+wb' \cot \alpha)\left[\frac{1+\sin \phi}{1-\sin \phi}\, e^{\pi \tan \phi}-1\right]$$

where, with respect to the supporting soil,

q=supporting power,
c=the unit cohesion,
ϕ=the angle of internal friction,
w=unit weight of undersoil,
$b'=\frac{a}{2}+b$ (see figs. 3 and 4), and
$\alpha=45°-\phi/2$.

SAMPLE CALCULATION OF SUPPORTING POWER GIVEN

As an example, suppose that q is to be determined for the supporting soil beneath a fill in which the dimension $a=b=20$ feet and which has a height of 20 feet, giving a value for $p=2{,}000$ pounds per square foot, the unit weight of fill material being 100 pounds per cubic foot. Suppose that the values c and ϕ for the supporting soil are 200 pounds per square foot and 5°, respectively. This value for c is less than $0.3\times2{,}000=600$ pounds per square foot. The value of α is $45°-\frac{5°}{2}=42.5°$ and $\cot \alpha=1.091$. Assume that w for the undersoil is 100 pounds per cubic foot. The value $\cot 5°=11.43$, $\sin \phi=0.087$ and $\tan \phi=0.0875$. Then by substitution in Prandtl's formula,

$$q=(200\times11.43+100\times30\times1.091)\left[\frac{1+0.087}{1-0.087}\, e^{\pi\times0.0875}-1\right]$$
$$=3{,}150 \text{ pounds per square foot.}$$

The factor of safety against complete failure of the supporting soil is

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[4] Principles of Soil Mechanics Involved in Fill Construction, by L. A. Palmer and E. S. Barber. Proceedings of the Highway Research Board, Annual Meeting, 1937.

[5] Prandtl's formula as originally derived pertains to uniform strip loading. The formula is adapted to the case of trapezoidal loading (fill) by reconstruction of the load diagram, changing the trapezoid (fig. 3) to a rectangle of equal area.

$$F=\frac{q}{p}=\frac{3,150}{2,000}=1.6.$$

The approximate working rule, that $0.3p$ should not exceed the unit cohesion c of the supporting soil, does not take into account the frictional resistance of the undersoil and the resistance offered by the weight of displaced undersoil when it fails. Prandtl's formula takes these factors into account. This approximate rule is usually well on the side of safety against ultimate failure if ϕ is greater than 10° and if the ground-water level is considerably below the surface of the supporting soil. Buoyancy produced by ground water diminishes the value of w in Prandtl's formula.

Only ultimate failure of the supporting soil is considered in the preceding example. It is entirely possible that even with a factor of safety of as much as 2 against ultimate failure, there may nevertheless be displacement of the undersoil to an extent such that the fill settles beyond the maximum allowable amount, and in this event it would be necessary to have a much higher factor of safety than 2. If a correlation between displacements of large soil masses on the one hand and deformations in laboratory samples on the other were obtained, it should be possible to take a laboratory value c' (rather than the ultimate value c) corresponding to the maximum allowable deformation in the supporting soil and refer the factor of safety to this value.

Example 2—Derivation of formulas for stresses beneath a trapezoidal load distribution.—With reference to the upper load diagram, *OBCD*, figure 2, Carothers has derived the formulas for stresses at any point (x, y, z) of the undersoil. For p_z at any point he obtains:

$$p_z=\frac{p}{\pi a}\left[a\beta+x\alpha-\frac{az}{R_3^2}(x-b)\right]\text{--------}(11)$$

If $b=a$, angle $\beta=0$, $R_3=R_2$, and formula (11) reduces to

$$p_z=\frac{p}{\pi a}\left[x\alpha-\frac{az}{R_2^2}(x-a)\right]\text{----------}(12)$$

which is the value of p_z at any point (x, y, z) in the undersoil beneath a triangular load (*OAB*, center diagram fig. 2) extending over a long distance in the Y direction. If p is now given a negative sign in formula (12), p_z becomes negative at the point. If, now, a strip of uniform pressure, p_0, is superimposed on the negative triangular load, *OAB* (see lower diagram of fig. 2), it is possible to derive the expression for p_z at the point (x, y, z) as produced by the trapezoidal load, *OBCD*. Thus, for the uniform pressure, p_0, over the strip of width a and represented by the load diagram *OACD*, lower diagram of figure 2,

$$p_z=\frac{p_0}{\pi a}(a\alpha+a\sin\alpha\cos 2\beta)\text{-------}(13)$$

If the expression on the right of formula (12) is now subtracted from the expression on the right of formula (13), there results,

$$p_z=\frac{p_0}{\pi a}(a\alpha+a\sin\alpha\cos 2\beta)-\frac{p}{\pi a}\left[x\alpha-\frac{az}{R_2^2}(x-a)\right]\text{--}(14)$$

which is the expression for p_z at any point (x, y, z) in the undersoil resulting from the trapezoidal load, *OBCD*, at the surface. In a similar manner it is found that

$$p_x=\frac{p_0}{\pi a}(a\alpha-a\sin\alpha\cos 2\beta)-\frac{p}{\pi a}\left[x\alpha+\frac{az}{R_2^2}(x-a)+2z\log_e\frac{R_2}{R_1}\right]\text{-}(15)$$

$$s_{xz}=\frac{p_0}{\pi a}(a\sin\alpha\sin 2\beta)+\frac{p}{\pi a}\left[z\alpha-a\frac{z^2}{R_2^2}\right]\text{-}(16)$$

and

$$s_{\max.}=\frac{1}{\pi a}\left\{\left[p_0a\sin\alpha\cos 2\beta+p\frac{az}{R_2^2}(x-a)+p_z\log_e\frac{R_2}{R_1}\right]^2+\left[p_0a\sin\alpha\sin 2\beta+p\left(z\alpha-a\frac{z^2}{R_2^2}\right)\right]^2\right\}^{1/2}\text{-------}(17)$$

By substitution of various values for α, β, R_1, R_2, x, and z in formula (17), it is found that the greatest value of $s_{\max.}$ is found at the point O, figure 5, and that this value is equal to $\frac{p_0}{\pi}$ where p_0 is the maximum applied pressure.

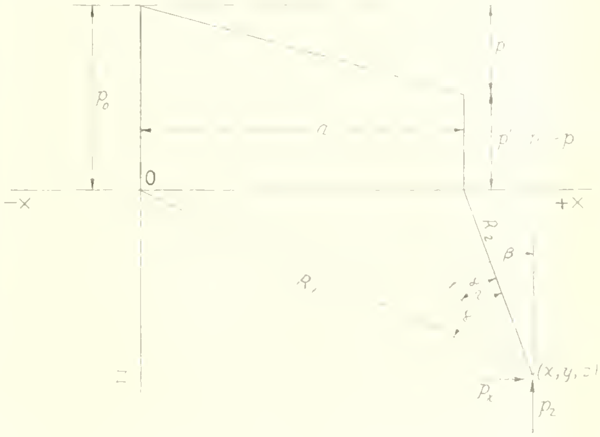

FIGURE 5.—TRAPEZOIDAL LOAD DISTRIBUTION DIAGRAM. SEE FORMULAS (14), (15), (16), AND (17).

CONSIDERATION GIVEN TO EARTH PRESSURES TENDING TO PRODUCE ROTATING MOMENT

A trapezoidal load distribution, as illustrated in figure 5, results whenever a rotating moment is applied to a retaining wall or abutment, and in this figure the point, O, may be considered as at the toe. There is no danger of failure of the undersoil if the cohesion of the supporting soil corresponding to the maximum allowable movement of the wall or abutment exceeds $\frac{p_0}{\pi}$. If the cohesion is less than this greatest shearing stress, it does not follow that failure of the undersoil will result although a plastic region at the toe is likely to be developed, and in this case the stress relations as given by formulas (14), (15), (16), and (17) no longer apply. The whole system of stress distribution is then changed. Prior to further consideration of this subject, it is necessary to consider the earth pressures tending to produce the rotating moment.

There are two principal kinds of soil deformation caused by load: (1) Volume change; and (2) distortion resulting from shear. Volume change results when a compressible soil stratum is consolidated by causing water to escape from the stratum under pressure into one or more permeable strata (drainage courses) bounding the compressible soil layer. On the other hand, it is assumed that distortion alone produces no volume change in a soil. One of the basic principles of soil mechanics has been that a soil maintained at a constant moisture content is incompressible. This could be true, of course, if the voids are completely filled with water and if each individual soil particle is assumed to be capable of a change in shape but not in volume, in which case the value of Poisson's ratio would be $\frac{1}{2}$.

The magnitude of distortion caused by shearing stresses depends on the magnitude of the stresses and the shearing resistance of the soil. It may proceed slowly and cause a slow subsidence of the supporting soil, or it may occur very quickly as in a landslide. The relatively slow displacement may cause failure of the structure without developing a surface of failure or shear in the supporting soil. The allowable deformation depends on the structure, and it may be considerably less than the magnitude of the deformation or distortion that is characteristic of the yield value or ultimate shearing resistance of the soil. Laboratory tests can provide the design engineer with a complete shearing stress-deformation relationship for the soil in question, and this, together with a knowledge of the earth stresses, should be very useful information.

At ultimate failure the shearing stress at any point on the surface of failure is equal to the shearing resistance of the soil at that point, and Coulomb's formula expressing this equality is

$$s=c+p_n \tan \phi \text{ - - - - - - - - - - - - - - } (18)$$

where s is the total unit shearing resistance and c the cohesion per unit of area. Cohesion may be only a part of the shearing resistance. If friction is developed, the other part is $p_n \tan \phi$ where ϕ is the angle of internal friction (limiting value of the angle of obliquity) and p_n is the pressure normal to the tangent plane through any given point on the surface of shear. Both c and ϕ are limiting values in this formula, that is, they are values which obtain at the instant of sudden and complete failure of the earth mass

If the shearing stress in the soil mass equals half the shearing resistance, s, then the factor of safety insofar as the soil is concerned is 2. However, for the structure supported by this soil mass, this number may be meaningless, c and ϕ being ultimate values, and it is preferable to take values of c and ϕ that correspond to the maximum allowable soil deformation for the particular structure. In formula (18) it is assumed that the normal pressure p_n is transmitted solely from grain to grain of soil and that no part of p_n is hydrostatic pressure. The conditions that exist when a portion of p_n is carried by water have been analyzed by Terzaghi.[6]

MOHR'S GRAPHICAL METHOD USEFUL IN PROBLEMS OF PLANE STRAIN

For any material in which ϕ is zero, the shearing strength is not increased by pressure. Thus, a cylindrical specimen of such material, subjected to a load in the direction of its axis, would have theoretically the same shearing strength for a lateral (hydraulic) pressure of zero as for one of considerable magnitude. If ϕ is not zero, the shearing strength increases with increasing lateral pressure, p_h. In a cylinder of homogeneous material, the displacements in any two or more different vertical planes through the longitudinal axis are the same. Points in any such vertical plane remain in that plane during the period of stress. Hence, the compressed cylinder is treated as though it were another case of plane strain. For a graphical analysis of the stresses in such a system, Mohr's diagram is very useful. The graphical method is illustrated in its simplest form in figure 6, and the following procedure enables one to compute the normal stress, p_θ, and the shearing stress, s_θ, for any plane, θ, through the stressed cylinder, p_v and p_h being the only applied stresses.

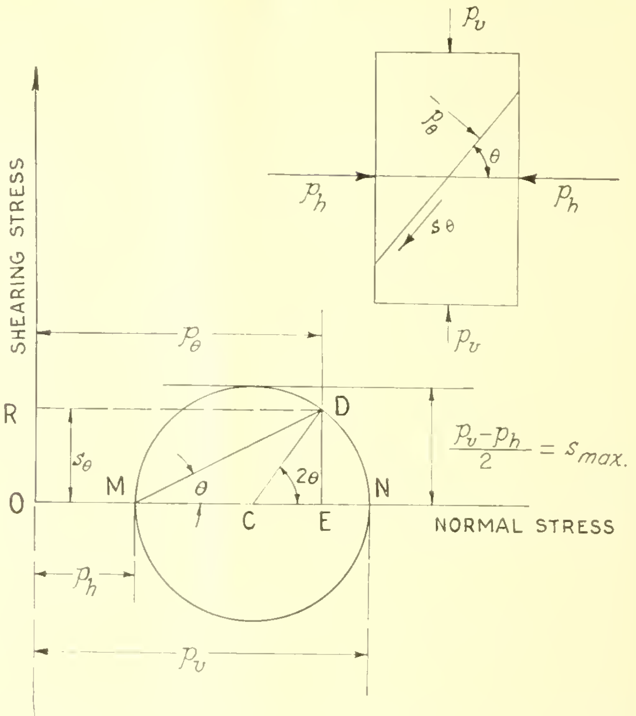

FIGURE 6.—GRAPHICAL ANALYSIS OF STRESSES IN A COMPRESSED CYLINDER, USING MOHR'S CIRCLE OF STRESS.

Procedure for computing p_θ and s_θ.—

1. On the normal stress axis, figure 6, lay off to scale the distances, OM and ON, proportional to p_h and p_v, respectively.
2. Draw a circle around the diameter, MN.
3. The normal stresses, p_v and p_h, are principal stresses. There are no shearing stresses on the planes (principal planes) to which these stresses are perpendicular. The maximum ordinate to any point on the circle is the radius of the circle. The diameter$=MN=p_v-p_h$, and the radius is $\frac{p_v-p_h}{2}=s_{\max}$.
4. A line drawn from M to any point, D, on the circle represents a plane through the center of the cylinder, inclined at an angle, θ, with the horizontal. There are an infinite number of planes through the center of the cylinder and a corresponding infinite number of points on the circle. The normal stress on the plane, θ, through the cylinder is the horizontal distance, RD, in the stress diagram,

$$RD=OE=OM+MC+CE=OM+MC+CD \cos 2\theta$$

$$=p_h+\frac{p_v-p_h}{2}+\left[\frac{p_v-p_h}{2}\right] \cos 2\theta$$

Hence

$$p_\theta=\frac{p_h+p_v}{2}+\left[\frac{p_v-p_h}{2}\right] \cos 2\theta \text{ - - - - - - } (19)$$

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[6] The Shearing Resistance of Saturated Soils and the Angle between the Planes of Shear, by Dr. Charles Terzaghi, vol. I, pp. 54 to 56, inclusive, Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, 1936.

For $\theta=0$, $p_\theta=p_v$ and for $\theta=45°$, $p_\theta=OC=\frac{p_v+p_h}{2}$. For $\theta=90°$, $p_\theta=p_h$.

5. The shearing stress, s_θ, on the plane, θ, is the ordinate value, DE. That is,

$$s_\theta=DE=CD\sin 2\theta$$

or

$$s_\theta=\left[\frac{p_v-p_h}{2}\right]\sin 2\theta \qquad (20)$$

For $\theta=45°$, s_θ is a maximum and equal to $\frac{p_v-p_h}{2}$. For $\theta=90°$, $s_\theta=0$ and for $\theta=0$, $s_\theta=0$. Hence on the two mutually perpendicular planes, $\theta=0$ and $\theta=90°$, there is no shearing stress, p_v being perpendicular to $\theta=0$ and p_h to $\theta=90°$.

PLANE WHERE SHEARING RESISTANCE EQUALS SHEARING STRESS DETERMINED GRAPHICALLY

In the stressed cylinder, p_v is the same on all horizontal planes and p_h is the same on all vertical planes. Hence p_θ and s_θ have constant values on one and the same plane. This condition does not exist in an earth mass where the principal normal stresses vary both in direction and magnitude from one point to another. Hence in an earth mass the Mohr's diagram must be considered with reference to stresses all in the immediate neighborhood of a particular point.

The graphical procedure above described is with reference to stress and without regard to the resistance of the material to stress; and no elastic constants are considered, since the problem is one of plane strain. If the angle, ϕ, characteristic of the material, is not zero and if a cylinder of the material fails in shear under a compression, p_v, and a lateral pressure, p_h, then if p_h is increased to p_h', the compressive strength, p_v, will be increased correspondingly to a value, p_v'. This condition may be represented by two stress circles (see fig. 7), the first having a diameter equal to p_v-p_h and the second a diameter equal to $p_v'-p_h'$. In both instances, failure occurs theoretically on that plane, θ, where the shearing resistance equals the shearing stress according to formula (18). The graphical procedure of determining this plane is as follows (see fig. 7):

1. Describe circles around the diameters, p_v-p_h and $p_v'-p_h'$. The distances, OM and OM', represent p_h and p_h', respectively.

2. Draw a straight line tangent to both circles. Draw the lines, MT and $M'T'$, to the points of tangency. These two lines represent the most dangerous planes and are called the planes of failure. Both are inclined at the same angle, α, with the horizontal.

3. The tangent line called "Mohr's envelope" is inclined at the angle, ϕ, to the horizontal. Its equation is formula (18),

$$s=c+p_n\tan\phi,$$

c and ϕ being ultimate values. At the origin, $p_n=0$ and the shearing resistance at zero normal pressure equals the cohesion, c. The intercept of the envelope line with the shearing stress axis is c and its slope is $\tan\phi$. From geometry, $2\alpha=90°+\phi$ or $\alpha=45°+\frac{\phi}{2}$. If the material is such that $\phi=0$, the most dangerous planes are inclined at 45° to the horizontal.

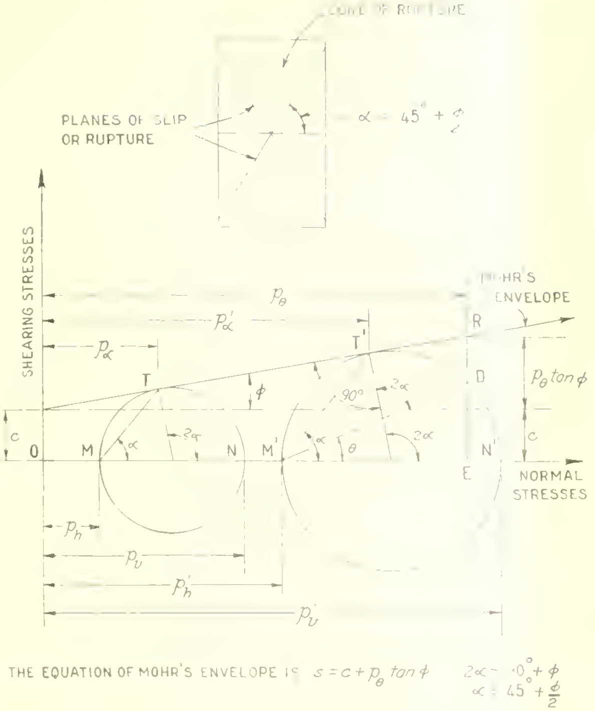

FIGURE 7.—GRAPHICAL ANALYSIS FOR DETERMINING PLANE OF FAILURE.

4. The shearing stress on any plane, for example the plane, $M'D$ on the larger circle, is obtained from formula (20). On the plane, $M'D$, the shearing stress is the ordinate ED, and the shearing resistance on this plane is $ER=c+p_\theta\tan\phi$ where p_θ is the pressure normal to the plane, $M'D$. The shearing resistance is greater than the shearing stress on this plane, and this condition exists on all planes except the most dangerous ones, where, according to Mohr's theory, the shearing stress equals the shearing resistance. On the plane, $M'T'$, the shearing resistance, s, is equal to $c+p'_\alpha\tan\phi$.

In accordance with this theory, failure may be by fracture, as in the crushing of a concrete cylinder, or it may be evidenced by plastic flow, in which case the dangerous planes are planes of "slip" or flow. When plastic yield occurs, helical slip lines (Lüders lines) are often visible on the exterior surface of the cylinder. Visual observations of Lüders lines and fractured surfaces in stressed cylinders of homogeneous materials tend generally to substantiate Mohr's theory, which is essentially a graphical representation of Coulomb's law, formula (18).

EARTH PRESSURES COMPUTED WITH THE AID OF MOHR'S THEORY

It has been previously indicated in this paper that in considering the supporting power of soils conservative values for cohesion and friction (that is, values less than the ultimate) should be taken. Thus the unit cohesion of the undersoil below a fill should not be exceeded by the greatest shearing stress, the unit cohesion in this case being that corresponding to the maximum allowable deformation and therefore less than the ultimate value, symbolized by the letter, c. In the development of Mohr's theory of failure and Coulomb's formula only the ultimate values, c and ϕ, are considered.

There are instances, however, when it is on the side of safety to use only the ultimate values, c and ϕ, in computations. An example is the computation of passive earth pressure against an abutment or retaining wall. The formula for the passive earth pressure is

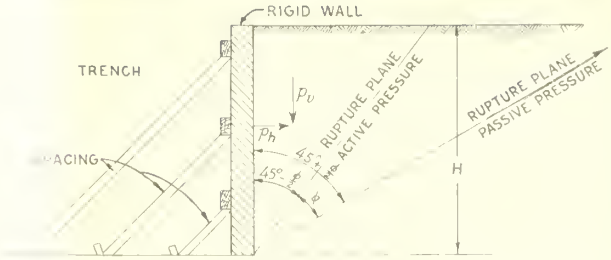

FIGURE 8.—SCHEMATIC DIAGRAM SHOWING EARTH MASS, RETAINING WALL, AND RUPTURE PLANES.

$$p\ (\text{passive}) = p_1 \tan^2\left(45° + \frac{\phi}{2}\right) + 2c \tan\left(45° + \frac{\phi}{2}\right)$$

where p (passive) and p_1 are principal stresses. It is not safe to underestimate the magnitude of p (passive), which, as seen in this formula, increases as c and ϕ increase and which is produced by the expansion of soil back of a retaining wall. This formula and the one for active earth pressure are developed by the following procedure.

Consider a level earth mass of cohesive and homogeneous soil of great depth and extent. It is assumed that it is possible to cut out a trench of given depth and of great length in the soil mass without any disturbance of the remaining earth. It is next supposed that a rigid wall with bracing is erected (still without disturbing the soil) along the entire length of a side of the perpendicular embankment (see fig. 8). The unit weight of earth is taken as w and if the vertical distance from the surface to a small element of earth touching the wall is H', then the vertical pressure, p_v, on the element equals wH', assuming zero friction between earth and wall. To this vertical pressure there corresponds a horizontal pressure, p_h, and it is assumed that the values, p_v and p_h, have remained unchanged during the construction of the rigid wall and bracing.

If the wall remains unyielding, the ratio, $\frac{p_h}{p_v}$, is known as the coefficient of earth pressure at rest. In figure 9, the stresses on the element adjacent to the wall and at a depth, H', are represented by circle (1), the circle for earth pressure at rest. It is assumed that the stresses are transmitted solely from grain to grain of soil. Terzaghi[7] has estimated the coefficient of earth pressure at rest as being within the limits, 0.70 to 0.75, in cohesive soils and ranging from 0.40 to 0.45 in sands. Various authors have expressed doubt as to the constancy of this ratio throughout a considerable depth of homogeneous soil. With reference to figure 8, where for the unyielding wall $\frac{p_h}{p_v} = K$, a constant, the total pressure against a vertical strip of unit width of the wall is $P_h = \frac{Kp_vH}{2}$ where H is the height of the wall. If $K = 0.70$, $H = 20$ feet, and $w = 100$ pounds per cubic foot, then

$$P_h = \frac{KwH^2}{2} = \frac{0.7 \times 100 \times 400}{2} = 14{,}000 \text{ pounds.}$$

SAMPLE CALCULATIONS OF EARTH PRESSURES GIVEN

Now assume that the wall yields and moves outward to an extent such that the soil mass back of it fails suddenly in shear. The weight of earth and therefore the value of p_v at any point along the wall are constant during this occurrence, but (fig. 9) the stress circle has grown and become tangent to the rupture line (Mohr's envelope). This has occurred by reason of a diminution in p_h and, with p_v constant, the diameter of the circle has increased from $p_v - p_h$ intermediate to $p_v - p_h$ minimum, and

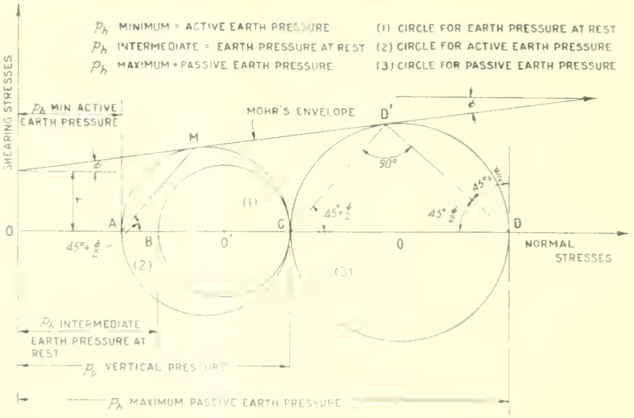
p_h MINIMUM = ACTIVE EARTH PRESSURE
p_h INTERMEDIATE = EARTH PRESSURE AT REST
p_h MAXIMUM = PASSIVE EARTH PRESSURE
(1) CIRCLE FOR EARTH PRESSURE AT REST
(2) CIRCLE FOR ACTIVE EARTH PRESSURE
(3) CIRCLE FOR PASSIVE EARTH PRESSURE
SHEARING STRESSES
p_h MIN ACTIVE EARTH PRESSURE
MOHR'S ENVELOPE
D'
ϕ
M
90°
c
(1)
$45°+\frac{\phi}{2}$
$45°\frac{\phi}{2}$
$45°+\frac{\phi}{2}$
0
A
B
0'
C
0
D
(2)
(3)
NORMAL STRESSES
p_h INTERMEDIATE EARTH PRESSURE AT REST
p_v VERTICAL PRESSURE
p_h MAXIMUM PASSIVE EARTH PRESSURE

FIGURE 9.—DIAGRAMS SHOWING ACTIVE AND PASSIVE EARTH PRESSURES AND EARTH PRESSURE AT REST.

$$p_h \text{ int.} = \text{earth pressure at rest,}$$

and

$$p_h \text{ min.} = \text{active earth pressure.}$$

Failure occurs along the surface AM (fig. 9), inclined at $45° + \frac{\phi}{2}$ to the horizontal or $45° - \frac{\phi}{2}$ to the vertical.

On the other hand, assume that enough lateral pressure is applied against the wall from the excavated side to cause the earth to shear and be displaced upward. The lateral pressure required to effect this displacement is equal to the passive earth pressure, p_h max. (see fig. 9). The magnitude of this pressure is denoted by the distance, OD, and failure occurs along a plane inclined at $45° + \frac{\phi}{2}$ to the vertical (DD', fig. 9).

The stress circles, (2) and (3) of figure 9, for the active and passive earth pressures, respectively, are redrawn in figure 10. It is then seen that in circle (2),

$$\cos\phi = \frac{O'M}{O'L} = \frac{\frac{p_v - p_h \text{ min.}}{2}}{O'B + BL} = \frac{\frac{p_v - p_h \text{ min.}}{2}}{c + \left[\frac{p_v + p_h \text{ min.}}{2}\right]\tan\phi}.$$

Then

$$\frac{p_h \text{ min.}}{p_v} = \frac{1 - \sin\phi}{1 + \sin\phi} - \frac{2c\cos\phi}{(1 + \sin\phi)p_v}$$

and hence

$$p_h \text{ min.} = p_v \tan^2\left(45° - \frac{\phi}{2}\right) - 2c\tan\left(45° - \frac{\phi}{2}\right) \text{---}(21)$$

With reference to circle (3), figure 10, it is evident that the relationship of p_v to p_h max. is the same as that of p_h min. to p_v, that is,

$$p_v = p_h \text{ max.} \tan^2\left(45° - \frac{\phi}{2}\right) - 2c\tan\left(45° - \frac{\phi}{2}\right)$$

or

$$p_h \text{ max.} = \frac{p_v}{\tan^2\left(45° - \frac{\phi}{2}\right)} + \frac{2c}{\tan\left(45° - \frac{\phi}{2}\right)}$$

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[7] A Fundamental Fallacy in Earth Pressure Computations, by Charles Terzaghi, publications from the Graduate School of Engineering, Harvard University, 1935–36, No. 182, Soil Mechanics Series No. 3.

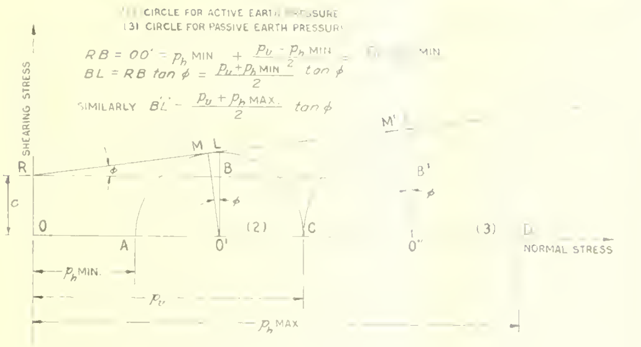

(2) CIRCLE FOR ACTIVE EARTH PRESSURE
(3) CIRCLE FOR PASSIVE EARTH PRESSURE
RB = OO' = p_h MIN + $\frac{p_v - p_h \text{ MIN}}{2}$ = MIN
BL = RB tan ϕ = $\frac{p_v + p_h \text{ MIN}}{2}$ tan ϕ
SIMILARLY B'L' = $\frac{p_v + p_h \text{ MAX.}}{2}$ tan ϕ
SHEARING STRESS
M'
M L
R
B
ϕ
B'
ϕ
c
ϕ
O
(2)
C
(3)
D
A
O'
O''
NORMAL STRESS
p_h MIN.
p_v
p_h MAX

Figure 10.—Diagrams Showing Active and Passive Earth Pressures.

and hence

$$p_h \text{ max.} = p_v \tan^2\left(45° + \frac{\phi}{2}\right) + 2c \tan\left(45° + \frac{\phi}{2}\right) \text{---}(22)$$

Take $c = 200$ pounds per square foot and $\phi = 20°$ with reference to the illustration, figure 8, then

$$p_h \text{ min.} = 0.490\ p_v - 280 \text{ pounds per square foot,}$$

and

$$p_h \text{ max.} = 2.039\ p_v + 571 \text{ pounds per square foot.}$$

For $w = 100$ pounds and $H = 20$ feet, the average p_v will be 1,000 pounds per square foot. Then, in summary, the total forces exerted against the wall by earth pressure at rest, active earth pressure, and passive earth pressure are:

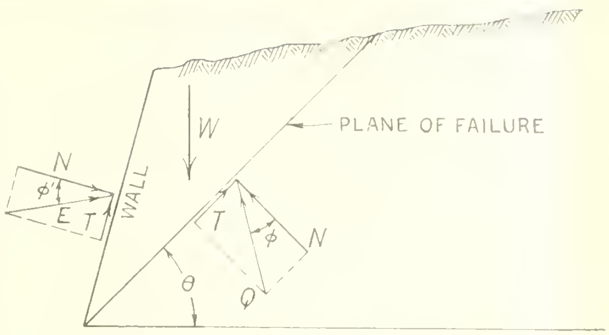
	Total lateral pressure on 1-foot vertical strip of wall
For earth pressure at rest---	$0.7 \times 1{,}000 \times 20 = 14{,}000$ pounds.
For active earth pressure---	$(490 - 280) \times 20 = 4{,}200$ pounds.
For passive earth pressure--	$(2{,}039 + 571) \times 20 = 52{,}200$ pounds.

Terzaghi[8] has shown that passive earth pressure may be exerted by clay soils against retaining walls. Such soils tend to expand and shrink noticeably, according to seasonal climatic variations. Hence, walls strong enough to withstand the pressure of a cohesionless backfill are gradually and intermittently pushed out of plumb by the swelling of a cohesive clay soil, characterized by low permeability and appreciable volume changes on wetting and drying. On swelling, the backfill exerts passive earth pressure against the wall.

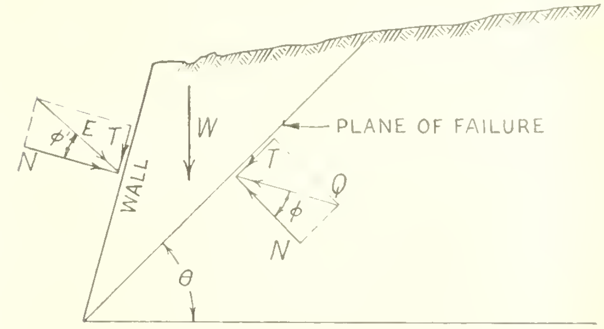
In the foregoing discussion and with reference to figures 8 and 9, the wall was assumed as vertical and parallel in direction to the principal plane to which p_h is normal. The surface of the backfill being level with the top of the wall and the friction between the wall and backfill material being taken as zero, there is no difficulty in locating the planes of failure for active and passive earth pressures. In both cases the resultant earth pressure (usually designated as E) is located at one-third the distance upward from the bottom of the cut, assuming that p_v increases directly with depth.

SHEARING PLANE DETERMINED BY GRAPHICAL METHODS

For the more general case the surface of the backfill is not level; there is a certain amount of friction between the backfill material and the wall; the wall is not parallel to a principal plane; and the force exerted against the wall is not horizontal. The analysis of the general case is that of Coulomb and this method deals with forces rather than stresses. One basic assumption is that the surface of failure is, for all practical purposes, a plane (actually it is more nearly an arc of a circle having a large radius of curvature). The solution is then briefly as follows:



ACTIVE EARTH PRESSURE



PASSIVE EARTH PRESSURE

N = NORMAL FORCE, T = FRICTIONAL FORCE
AND E AND Q ARE RESULTANT FORCES

Figure 11.—Force Diagrams for Active and Passive Earth Pressures.

In figure 11, let θ be the angle between the plane of failure and the horizontal, and W the weight of earth which slides when the earth fails. It is assumed that the magnitude of θ is such that the resultant force, E, against the wall is a maximum or, that if E is some function of θ, then $\frac{dE}{d\theta} = 0$. It is further assumed that ϕ', the angle of friction between the wall and the earth, is less than ϕ, the angle of internal friction of the earth.

Assume for the present (see fig. 11) that the positions of the failure planes and the angles ϕ and ϕ' are known. In the upper diagram of figure 11 for active earth pressure the earth moves downward over the sliding plane, whereas in the lower diagram for passive earth pressure movement of earth is upward over the sliding plane. The tangential forces, both along the wall and sliding plane, are therefore opposite in direction in the two cases. Hence the resultant force E against the wall acts below the normal to the wall in the upper diagram and above the normal to the wall in the lower diagram and likewise the relative positions of the resultant force Q and the normal force N on the plane of failure are different in the two cases.

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[8] The Mechanics of Shear Failures on Clay Slopes and the Creep of Retaining Walls, by Charles Terzaghi, Public Roads, vol. 10, No. 10, December 1929.

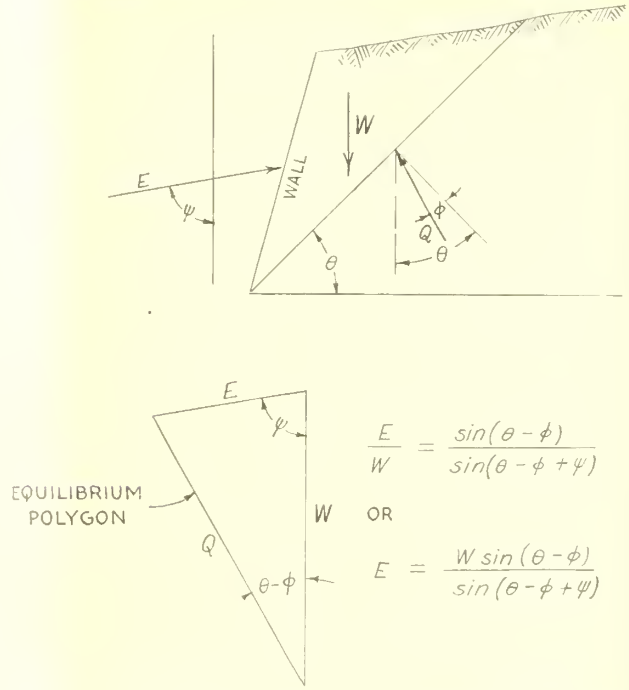


FIGURE 12.—FORCE DIAGRAM AND EQUILIBRIUM POLYGON FOR ACTIVE EARTH PRESSURE.

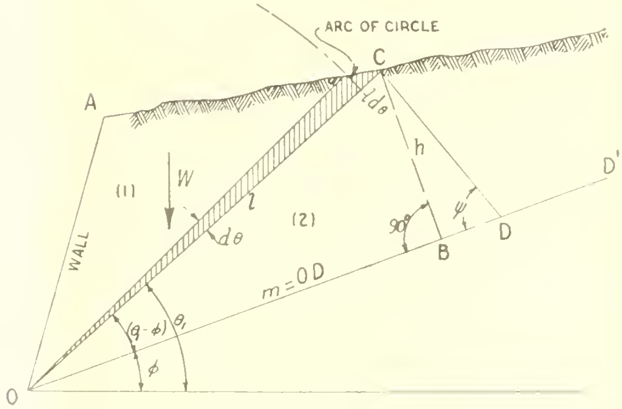


FIGURE 13.—DIAGRAM SHOWING METHOD OF LOCATING THE SLIDING PLANE FOR ACTIVE EARTH PRESSURE.

The position of the plane of failure cannot, however, be assumed. It must be found. For active earth pressure, let ψ be the angle between the direction of E and the vertical (see fig. 12). From the equilibrium polygon of forces, as shown in this figure,

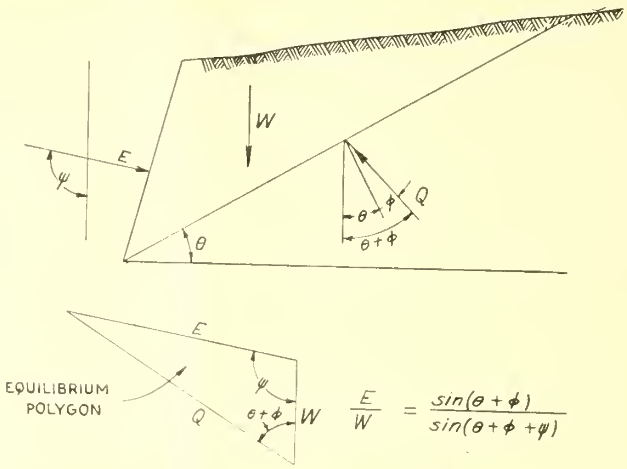
$$E=\frac{W\sin(\theta-\phi)}{\sin(\theta-\phi+\psi)} \qquad (23)$$

and, for E to be a maximum,

$$\frac{dE}{d\theta}=0=\frac{\sin(\theta-\phi+\psi)\left[\frac{dW}{d\theta}\sin(\theta-\phi)+W\cos(\theta-\phi)\right]-W\sin(\theta-\phi)\cos(\theta-\phi+\psi)}{\sin^2(\theta-\phi+\psi)}$$

That is,

$$W[\sin(\theta-\phi+\psi)\cos(\theta-\phi)-\sin(\theta-\phi)\cos(\theta-\phi+\psi)]$$
$$=-\left[\frac{dW}{d\theta}\right]\sin(\theta-\phi)\sin(\theta-\phi+\psi).$$



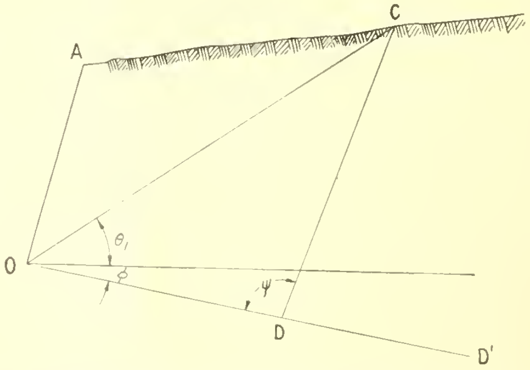


FIGURE 14.—DIAGRAM SHOWING METHOD OF LOCATING THE SLIDING PLANE FOR PASSIVE EARTH PRESSURE.

Therefore

$$W=\left[-\frac{dW}{d\theta}\right]_{\theta_1}\frac{\sin(\theta_1-\phi)\sin(\theta-\phi+\psi)}{\sin\psi} \qquad (24)$$

where θ_1 is the value of θ for which E is a maximum.

In figure 13 the line $OC=l$ and the area of the shaded triangle is approximately $\frac{1}{2}l^2d\theta$. The weight of earth in the shaded triangle is

$$-(dW)_{\theta_1}=\frac{1}{2}wl^2(d\theta)_{\theta_1}$$

or

$$\left[-\frac{dW}{d\theta}\right]_{\theta_1}=\frac{1}{2}wl^2 \qquad (25)$$

Draw OD', inclined at the angle ϕ with the horizontal, and draw CD to intersect OD' at the angle ψ. Drop the perpendicular, h, from C to OD'. Then

$$\frac{m}{l}=\frac{\sin[\pi-(\theta_1-\phi+\psi)]}{\sin\psi}=\frac{\sin(\theta_1-\phi+\psi)}{\sin\psi}$$

where $m=OD$. Furthermore, $\frac{\sin(\theta_1-\phi)}{\sin 90^\circ}=\frac{h}{l}$ and substituting this value, together with $\frac{m}{l}$ and $\frac{1}{2}wl^2$ from formula (25) in formula (24), there is obtained

$$W=\frac{1}{2}wl^2\times\frac{h}{l}\times\frac{m}{l}=\frac{whm}{2} \qquad (26)$$

Hence W is equal to the area $COD \times w$ (fig. 13). Obviously for a maximum active earth pressure, the area $OACD$ (fig. 13) must be divided equally so that the area of the triangle AOC = area of COD, and CD may be moved parallel to itself until this condition obtains, and triangle (1) is equal in area to triangle (2) (fig. 13).

For determining passive pressures, except for minor changes, the procedure is the same as that for active pressures. The slight differences are shown in figure 14. Here the line OD' is drawn below the horizontal rather than above it, the angle between OD' and the horizontal being ϕ in both active and passive pressure diagrams. For passive earth pressure (see fig. 14),

$$E = \frac{W \sin(\theta + \phi)}{\sin(\theta + \phi + \psi)} \quad \text{----------} \quad (27)$$

Setting $\frac{dE}{d\theta} = 0$ as before and designating the value of θ for which E is a maximum as θ_1,

$$W = \left[-\frac{dW}{d\theta} \right]_{\theta_1} \frac{\sin(\theta_1 + \phi) \sin(\theta_1 + \phi + \psi)}{\sin \psi} \quad \text{--} \quad (28)$$

and except for the sign of ϕ, formula (28) is the same as (24), which is for active earth pressure. By moving the line CD (which makes an angle, ψ, with OD'), parallel to itself, its position when the area of triangle OAC = area OCD, meets the requirement that E is a maximum and therefore that OC is the sliding plane.

MAXIMUM EARTH STRESSES MUST BE PROVIDED FOR IN DESIGN

There are various graphical procedures [9] used in computing active and passive earth pressures and their distributions against retaining walls. The procedures are merely simplified forms of the foregoing combined analytical and graphical procedures. In addition to simplification, it is on the side of safety to take ϕ' as zero. The pressure distribution against the wall is triangular, increasing from zero at the top to a maximum at the base. The resultant force (total pressure), E, is considered as applied at the intersection of the medians of this triangle, and this is one-third of the distance upward from the base of the fill.

Vertical pressure under an abutment.—Because of an overturning moment [10] caused by any of the three earth pressures (active, passive, or earth pressure at rest) the vertical pressure distribution under an abutment is in general trapezoidal (see fig. 5), the greatest pressure, p_0, being directly under the toe. As shown previously in this paper, if the cohesion of the supporting soil exceeds $\frac{p_0}{\pi}$ there is no danger of failure of the undersoil. On the other hand, if the cohesion is less than $\frac{p_0}{\pi}$, failure of the undersoil is not inevitable.

One may go farther in this case and apply the general method of Hogentogler and Terzaghi [11] in investigating the stability of the supporting soil. The analysis is illustrated in figure 15. In computing the greatest shearing stress at any point in the undersoil and also in connection with any computations of settlement of the abutment (fig. 15) resulting from slow consolidation of one or more compressible soil strata below the abutment, it is necessary to determine the load diagram (such as $AA'B'B$, upper diagram of fig. 15) and use it without transformation. However, in the following investigation of the possibility of the development of surface failure in the supporting soil (ACD, fig. 15) it is on the side of safety to consider the load transmitted by the wall or abutment as uniformly distributed along the plane, AB. For since the wedge, ABC, tends to move downward along the plane, AC, and push out and upward the wedge, BCD, along the plane, CD, it is obvious that an excess of pressure at B over that at A offers resistance to the forward movement of the wedge, ABC.

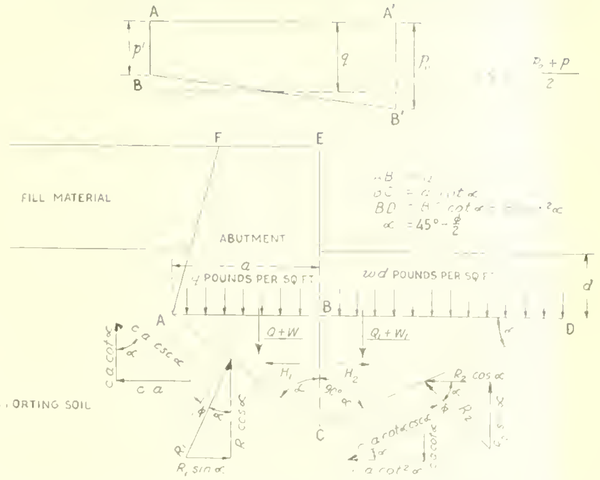


FIGURE 15.—DIAGRAM SHOWING METHOD OF COMPUTING SUPPORTING POWER OF SOIL BELOW BRIDGE ABUTMENT.

FORCES COMPUTED FOR CONDITIONS OF EQUILIBRIUM

With reference to figure 15, the average pressure along AB is $\frac{p' + p_0}{2} = q$, and it is assumed that this acts uniformly over AB. The vertical section is supposed to be of unit thickness in the direction perpendicular to the plane of the paper. Neglecting frictional resistance along AB and the shearing resistance of the surcharge of thickness, d, above the plane, BD, is again on the side of safety and simplifies the problem. Use is now made of the principles of active and passive earth pressures. The angle $\alpha = 45° - \frac{\phi}{2}$. The shearing plane for active earth pressure, as already shown, is inclined at an angle of $45° - \frac{\phi}{2}$ with the vertical, and the shearing plane for passive earth pressure is inclined at $45° + \frac{\phi}{2} = 90° - \alpha$ to the vertical (see figs. 8 and 9). Assume that the unit weight, w, of the undersoil is constant throughout the region indicated in figure 15. Computations will be made in terms of forces and not stresses.

The total vertical load on AB, of width a, is equal to $Q = qa$. The weight of the wedge, ABC, is $\frac{w(AB \times BC)}{2}$ or weight $= \frac{wa^2 \cot \alpha}{2} = W$. Then the total vertical force acting on the wedge, ABC, is $aq + \frac{wa^2 \cot \alpha}{2}$

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[9] See for example, Notes on Soil Mechanics and Foundations, by Fred L. Plummer (Edwards Brothers, Inc., Ann Arbor, Mich.), pp. 128 to 130, inclusive. See also pl. No. 17, p. 20, booklet, Soil Stabilization, published by the American Road Builder's Ass'n, 1938.

[10] For methods of computing the overturning and resisting moments, see pl. No. 16 p. 19, booklet, Soil Stabilization, published by the American Road Builders' Ass'n., 1938.

[11] Interrelationship of Load, Road and Subgrade, by C. A. Hogentogler and Charles Terzaghi, PUBLIC ROADS, vol. 10, No. 3, May 1929.

This downward force is resisted by the vertical component of the cohesion along AC, which is $ca \cot \alpha$ and by the vertical component, $R_1 \cos \alpha$, where R_1 is the resultant of the normal and frictional forces on AC. Then for a condition of equilibrium,

$$R_1 \cos \alpha + ca \cot \alpha = aq + \frac{wa^2 \cot \alpha}{2}$$

or

$$R_1 \cos \alpha = aq + \frac{wa^2 \cot \alpha}{2} - ca \cot \alpha \quad (29)$$

Similarly for equilibrium $R_1 \sin \alpha - ca = H_1$ or

$$R_1 \sin \alpha = H_1 + ca \quad (30)$$

Dividing equation (30) by (29),

$$\tan \alpha = \frac{H_1 + ca}{aq + \frac{wa^2 \cot \alpha}{2} - ca \cot \alpha}.$$

Hence

$$H_1 = \frac{wa^2}{2} + aq \tan \alpha - 2ca \quad (31)$$

Now consider forces acting on the wedge, BCD. The weight of the surcharge on the plane, BD, is $wd \times BD$ or $wda \cot^2 \alpha = Q_1$. The weight of the wedge, BCD, is $\frac{w}{2} BC \times BD = \frac{1}{2} wa^2 \cot^3 \alpha = W_1$. The total downward force is then $Q_1 + W_1$ and this together with the vertical component of the cohesive resistance along CD must equal the upward component, $R_2 \sin \alpha$ for a condition of equilibrium to obtain. That is,

$$R_2 \sin \alpha = \frac{wa^2 \cot^3 \alpha}{2} + wda \cot^2 \alpha + ca \cot \alpha \quad (32)$$

Similarly,

$$R_2 \cos \alpha = H_2 - ca \cot^2 \alpha \quad (33)$$

Dividing equation (32) by (33) and solving for H_2,

$$H_2 = \frac{wa^2 \cot^4 \alpha}{2} + wda \cot^3 \alpha + 2ca \cot^2 \alpha \quad (34)$$

For equilibrium, $H_1 = H_2$ and from equations (31) and (34), therefore,

$$\frac{wa^2}{2} + aq \tan \alpha - 2ca = \frac{wa^2 \cot^4 \alpha}{2} + wda \cot^3 \alpha + 2ca \cot^2 \alpha$$

and hence

$$q = \frac{2c}{\tan \alpha \sin^2 \alpha} + \frac{wa}{2 \tan \alpha}\left[\frac{1}{\tan^4 \alpha} - 1\right] + \frac{wd}{\tan^4 \alpha} \quad (35)$$

where q is the bearing capacity or supporting power of the undersoil. For $\phi = 0$, as is the case in a purely cohesive soil, $q = 4c + wd$, which means that the supporting power is independent of a, the width of the bearing area. In a compression test of an unconfined cylinder of the purely cohesive soil, $p_h = 0$ and $p_v =$ compressive strength $= 2c$, since $c = \frac{p_v - p_h}{2} = \frac{p_v}{2}$. Hence for such a soil and with reference to figure 15, the supporting power, q, is equal to twice the compressive strength (as determined in an unconfined cylinder compression test) plus wd.

If the supporting soil is clay, ϕ is small and

$$q = 4c + wd \quad (36)$$

is a safe working formula. Therefore if the supporting undersoil is clay, the design engineer is first of all concerned with the relative magnitudes of the cohesion and $\frac{p_0}{\pi}$. If the cohesion is greater than $\frac{p_0}{\pi}$ (the cohesion being a conservative value based on the allowable displacement of the structure), the problem ends there. If the cohesion is less than $\frac{p_0}{\pi}$, then q must not exceed $4c + wd$ in any case, the value q being the average unit vertical load transmitted by the wall or abutment to the undersoil. For conditions in which ϕ is greater than zero, q may be computed directly from equation (35).

METHOD BASED ON PLANE STRAIN HAS LIMITATIONS

For a uniform surface load over a long strip of parallel sides, Prandtl's method [12] yields $q = 5.14c$ for a purely cohesive soil, $\phi = 0$, and for the same conditions the method of Krey [13] gives $q = 6.6c$.

The legitimate use of the formulas for plane strain in connection with abutment and retaining wall problems depends chiefly on the ratio of length to width of the footing. There have apparently been no rules as to procedures where this ratio is small. Furthermore, for a rectangular footing whose length is not great in comparison to its width, there are apparently no formulas for stresses other than the vertical one, p_z and the lateral ones p_x and p_y, to be found in current literature. It is interesting therefore to compute p_z for a relatively long footing both by the formula as applicable to the case of plane strain and by the formula for the case involving three dimensions. It should be noted that p_z is independent of elastic constants in either method of computation, and it is the only stress having this characteristic.

In order to indicate the limitations of the method based on plane strain, four different footings will be considered. These are:

No. 1	50 feet long, 8 feet wide.
No. 2	35 feet long, 10 feet wide.
No. 3	30 feet long, 10 feet wide.
No. 4	20 feet long, 10 feet wide.

It is assumed that there is a uniform load distribution of 3 tons per square foot on each of these four footings. The vertical pressure, p_z, at any point in the earth at a depth of z feet below the footing is then computed by two different methods. These are:

a. The method based on plane strain.

b. The method based on rectangles using Newmark's tables [14] for convenience.

Newmark's method is both convenient and precise. Another method has been shown in a previous publication [15] and in its use the rectangular footing is sub-

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[12] Über die Harte Plastischer Körper Nachrichten von der Gesellschaft der Wissenschaften zu Göttingen, Mathematisch Physikalische Klasse, pp. 74–85, 1920.

[13] Erddruck Erdwiderstand und Tragfähigkeit des Baugrundes, H. Krey, pp. 193 and 268.

[14] Simplified Computation of Vertical Pressures in Elastic Foundations, by Nathan M. Newmark, Circular No. 24, vol. 33, No. 4, Engineering Experiment Station, University of Illinois, Sept. 24, 1935.

[15] The Theory of Soil Consolidation and Testing of Foundation Soils, by L. A. Palmer and E. S. Barber, PUBLIC ROADS, vol. 18, No. 1, March 1937.

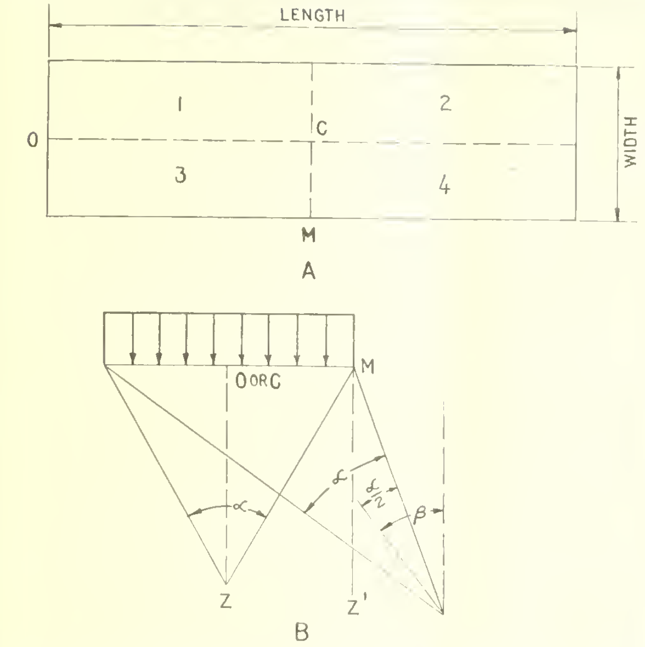

Figure 16.—Diagram Showing Values Involved in Computing the Normal Stress p_z.

divided into elements. The load on each element of area is then considered as a point load at the center of the element of area. The computation of p_z by either method, (a) or (b), involves no elastic constants. The formulas for p_z as developed by Boussinesq for a point load and as extended by others do not involve elastic constants.

With reference to figure 16A, p_z will be computed for depths of 5, 10, 15, and 20 feet along vertical lines through the points, *C*, the center of footing, *O*, the midpoint of an end, and *M*, the midpoint of a side. For the method of plane strain, the general formula is

$$p_z=\frac{p}{\pi}(\alpha+\sin\alpha\cos 2\beta)\text{ ....(37)}$$

where α and β are the angles shown in figure 16B. Along the vertical line *CZ*, $2\beta=0$ and formula (37) becomes

$$p_z=\frac{p}{\pi}(\alpha+\sin\alpha)\text{ ....(38)}$$

Along the vertical line, *OZ*, for points below *O* at the end of the footing, half the value of p_z as determined by formula (38) must be taken. For points on *MZ'* figure 16B, $2\beta=\alpha$.

Newmark's tables are based on values for p_z at points on a vertical line through a corner of rectangles of various sizes. Thus, the point, *C*, of figure 16A is a corner common to each of the four rectangles, 1, 2, 3, and 4, all of the same dimensions. The value p_z for a point on *CZ* is first obtained for one of these rectangles, and this value multiplied by 4 gives p_z (for a point on *CZ*) as resulting from the load on the entire footing. Similarly, the point, *O* is a corner, common to the two rectangles, 1 and 2, taken together and 3 and 4 taken together.

An example will clarify the two methods of computing p_z. It is desired to know p_z at 20 feet below the point, *C*, for footing No. 3, 10 feet by 30 feet, the unit load being 3 tons per square foot. The rectangles, 1, 2, 3, and 4 are each 5 feet by 15 feet,

$$\sin\alpha=2\sin\frac{\alpha}{2}\cos\frac{\alpha}{2}=2\left[\frac{5}{\sqrt{425}}\times\frac{20}{\sqrt{425}}\right]=\frac{200}{425}=0.471$$

and hence $\alpha=0.490$ in radians. Then by formula (38),

$$p_z=\frac{3}{\pi}(0.490+0.471)=0.92\text{ tons per square foot.}$$

From Newmark's tables, $m=\frac{5}{20}=0.25=$ the width of one of the rectangles, 1, 2, 3, or 4, divided by the depth, 20 feet, and $n=\frac{\text{length}}{\text{depth}}=\frac{15}{20}=0.75$. The "coefficient" corresponding to $m=0.25$ and $n=0.75$ is 0.05986. This is first multiplied by 4 for the 4 rectangles and then by 3 tons per square foot, that is,

$$p_z=0.05986\times 12=0.72\text{ tons per square foot.}$$

In this manner, the values for p_z in table 1 are computed for the footings, 8 by 50 feet, 10 by 35 feet, 10 by 30 feet, and 10 by 20 feet, each with a uniform load of 3 tons per square foot, and at various depths below the points *C*, *O*, and *M* (fig. 16).

Table 1.—*Values of p_z computed by methods based on plane strain and on rectangles, assuming a uniform loading of 3 tons per square foot*

Size of footing (feet)	Depth of point considered	Vertical pressure, p_z					
		Below point *C* by method based on—		Below point *O* by method based on—		Below point *M* by method based on—	
		Plane strain	Rectangles	Plane strain	Rectangles	Plane strain	Rectangles
	Feet	*Tons per sq. ft.*	*Tons per sq. ft.*	*Tons per sq. ft.*	*Tons per sq. ft.*	*Tons per sq. ft.*	*Tons per sq. ft.*
10 by 20	5	2.46	2.40	1.23	1.23	1.44	1.39
	10	1.65	1.44	.82	.81	1.23	1.05
	15	1.28	.94	.64	.57	1.02	.75
	20	.92	.57	.46	.40	.82	.50
10 by 30	5	2.46	2.44	1.23	1.23	1.44	1.43
	10	1.65	1.57	.82	.82	1.23	1.16
	15	1.28	1.02	.64	.60	1.02	.90
	20	.92	.72	.46	.44	.82	.64
10 by 35	5	2.46	2.45	1.23	1.23	1.44	1.43
	10	1.65	1.60	.82	.82	1.23	1.18
	15	1.28	1.12	.64	.60	1.02	.94
	20	.92	.77	.46	.44	.82	.69
8 by 50	5	2.22	2.22	1.11	1.11	1.40	1.39
	10	1.39	1.37	.69	.69	1.11	1.10
	20	.74	.69	.37	.37	.69	.64

RESULTS OBTAINED BY TWO METHODS COMPARED

For a given footing it is seen in table 1 that the difference in values for p_z as computed by the two methods increases with depth. The method based on plane strain is based on the assumption that the loaded foundation is of "infinite" length. Only finite lengths are considered in table 1 and for this reason the values obtained by the method based on rectangles are considered to be the correct ones.

In the light of present knowledge it is not possible to compute shearing stresses by the method based on rectangles but it is possible to do this by the method based on plane strain. As indicated previously it is necessary to know the shearing stresses and for this reason it is desirable to use the method based on plane strain when the error involved in so doing is not too large. Since it is possible to compute p_z for different points at a given depth by both methods, it should be reasonable to assume that if the values so computed are in fairly good agreement, then the analytical method illustrated in figure 15 and based on the assumption of plane strain conditions is applicable in any case when the failure plane (*AC*, fig. 15) does not extend below the depth in question.

In making use of table 1 it should be borne in mind that the ratio of width to length of a footing is the important consideration. Thus, any conclusion reached with respect to the footing that is 10 feet wide and 30 feet long is applicable also to a footing that is 20 feet wide and 60 feet long, or 5 feet wide and 15 feet long, etc.

Another point to be considered is the following: In figure 15, illustrating failure under an abutment, the soil moves out in but one direction, that is, along plane *CD* which is on the open side of the abutment or retaining wall. The depth *BC* in this case is the width, a, multiplied by $\cot\left(45^\circ-\frac{\phi}{2}\right)$. Thus, for $\phi=0$, *BC* (the depth to which the failure plane *AC* extends) is equal to a. On the other hand the supporting soil under a bridge pier is free to move out in two directions rather than but one. Then from symmetry the depth to which the failure plane extends under a pier is $\frac{a}{2}\cot\left(45^\circ-\frac{\phi}{2}\right)$ where a is the width of the pier.

To illustrate, if the 10 by 30-foot footing of table 1 is a bridge pier and if $\phi=0$, the plane of failure extends to a depth of 5 feet below the pier. For points at this depth it is seen in table 1 that the values for p_z computed by the two methods is in good agreement. On the other hand if the base of an abutment has these same dimensions and ϕ of the supporting soil is zero, then the failure plane extends downward to a depth $=a=10$ feet and for this depth there is greater divergence in the two sets of computed values of p_z. The more serious consideration then is that of the abutment or retaining wall where the failure plane extends to twice the depth that obtains in the case of a pier.

As ϕ increases, $\cot\left(45^\circ-\frac{\phi}{2}\right)$ increases and the depth (*BC* figure 15) $=a\times\cot\left(45^\circ-\frac{\phi}{2}\right)$ increases. A few computed values are as follows:

φ, degrees	*Depth of failure plane*
0	a
10	1. 19a
20	1. 43a
22	1. 48a
30	1. 73a
38	2. 05a

For $\phi=0$ and for a bridge pier of the dimensions shown in table 1, the failure plane extends down to 4 feet below the last footing and to 5 feet below the other three. Since at this depth the agreement in the two sets of computed p_z values is good in all cases, it is concluded that for any bridge pier having the same relative dimensions as those shown in table 1, the analytical method, assuming the method based on plane strain, is well warranted if $\phi=0$. If the first footing is excepted the same conclusion is reached in the case of abutments or retaining walls of the same relative dimensions. In this case the plane of failure extends down to 10 feet below the first three footings. The divergence in p_z, 1.57 to 1.65 tons per square foot below point *C* and 1.16 to 1.23 below point *M* (fig. 16) at 10 feet below the 10×30-foot footing is not considered as being serious.

By the same reasoning it is concluded that if the failure plane does not extend below 1½ times the width of the base of the abutment or retaining wall, the analytical method assuming plane strain conditions is warranted for any footing having a length-to-width ratio of 3 or greater. This includes all types of supporting soils having a value of ϕ of about 22°. The value of ϕ for most clays seldom exceeds 20° and this type of soil is usually the most dangerous insofar as supporting power is concerned. Values of ϕ exceeding 20° are characteristic of soils of considerable sand content. The value of ϕ for sands ranges usually from 35 to 40°, an approximate average value being 38° for which the failure plane extends to a depth of 1.73a for an abutment and to 0.87a for a pier.

CONCLUSIONS

The following conclusions are believed warranted:

1. The analytical method, assuming plane strain conditions, is applicable for all bridge piers of the relative dimensions given in table 1 (length-to-width ratio of 2 or greater) and for all values of ϕ ranging from 0 to values characteristic of sand.
2. The same analytical method is warranted for bridge abutments and retaining walls when the length-to-width ratio of the base is 3 or greater, ϕ having any value ranging from 0 to 22°. This includes practically all clays which present the major problem with respect to supporting power.

In the absence of percolating water or hydrostatic uplift, sands do not generally present a serious problem.

It is possible to check the accuracy of the assumption of conditions of plane strain by computations of the stress, p_z. It is not possible to check the assumption by computations of values of all other stresses. On account of this fact there is no check on the reasonableness of the assumption of conditions of plane strain other than the agreement of p_z values as computed by the method based on rectangles and by the method based on plane strain. A study of past and current soil mechanics literature does not indicate any means of computing by methods using three dimensions the shearing stresses at points below a rectangular loaded area.

The problem has, however, been solved in part for a loaded circular area [16] and Jürgenson [17] computes the stresses below a square footing by assuming a circle of equivalent area. For the loaded circular area, the analysis is limited to merely computing the stresses at any point below. Thus, it is known that the greatest shearing stress is equal to $\frac{p}{\pi}$, where p is the unit load (when the load distribution is uniform) and this shearing stress exists at all points immediately beneath the perimeter.

For a circular footing there is as yet no solution to the problem of determining the supporting power of the undersoil when c, the unit cohesion, is less than $\frac{p}{\pi}$. Nadai [18] shows that on the basis of experiment the forcing of a punch of cylindrical cross section into a plastic metal produces flow figures in the shape of logarithmic spirals, and the same phenomenon is observed in the heads of boilers subjected to plastic bending under an axially symmetrical stress field. There is, however, no adequate theory to account for this occurrence.

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[16] Treatise on the Mathematical Theory of Elasticity (see p. 190), by A. E. H. Love, 4th ed. Cambridge, University Press, 1934.

[17] The Application of Theories of Elasticity and Plasticity to Foundation Problems, by Leo Jürgenson, Journal of the Boston Society of Civil Engineers, vol. 21, No. 3, July 1934.

[18] Plasticity, A. Nadai, p. 228, McGraw-Hill Book Co., 1931.

Only rigid footings and static loads have been assumed and considered in this paper. The case of flexible footings presents complicated factors which for the most part have not as yet been satisfactorily accounted for. The problem of dynamic loading still awaits a theoretical solution substantiated by ample experimental data.

THE COVER PICTURE

The value of preserving trees and protecting them from damage during highway construction is shown by the cover picture. This section of U. S. Highway No. 1, near Brunswick, Maine, passes through a pine forest known as "The Bowdoin Pines."

Modern highways are designed so as to preserve the natural beauty of the roadside without sacrifice of service or utility. This has been successfully done on the highway shown. Here the grade of the highway is at approximately the same elevation as the surface of the adjacent ground. Note how the undergrowth screens the bases of many of the large trees, and how inconspicuous the poles and wires become.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF NOVEMBER 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 2,882,571	$ 1,347,485	82.8	$ 8,249,172	$ 4,113,175	354.9	$ 2,376,954	$ 1,187,935	67.2	$ 2,102,846
Arizona	1,906,924	1,483,382	107.5	889,645	574,320	24.9	333,503	140,345	25.8	1,106,569
Arkansas	938,120	933,384	70.0	3,032,208	3,028,471	184.9	542,108	542,003	39.1	723,592
California	6,503,704	3,533,424	132.4	7,793,061	4,153,140	141.9	1,671,940	884,689	45.3	1,279,156
Colorado	1,974,682	1,065,932	73.2	3,304,793	1,744,317	102.0	810,660	450,740	21.8	1,470,339
Connecticut	183,510	89,250	2.4	1,190,108	583,050	12.3	244,930	120,990	2.0	1,525,005
Delaware	446,780	221,184	13.6	711,750	353,895	9.9	265,351	128,530	6.6	1,137,482
Florida	1,184,100	592,050	36.9	3,589,938	1,794,969	60.0	277,000	138,500	3.9	2,843,195
Georgia	3,762,708	1,839,849	195.1	5,444,474	2,722,237	253.8	809,540	404,770	57.1	5,365,212
Idaho	2,024,215	1,199,579	194.5	1,101,321	657,611	33.6	99,697	59,667	5.4	1,097,803
Illinois	4,872,534	2,432,095	156.6	11,106,011	5,551,699	239.6	1,242,791	621,350	35.7	2,214,901
Indiana	4,213,808	2,101,210	117.5	3,836,480	1,918,240	76.5	1,659,490	821,699	32.2	2,191,813
Iowa	6,271,239	2,922,519	205.2	5,284,744	2,350,283	178.3	718,778	336,500	24.2	83,747
Kansas	4,074,179	2,017,365	638.7	4,294,889	2,122,392	217.0	3,397,638	1,697,819	164.3	2,453,720
Kentucky	5,042,152	2,505,062	183.5	3,225,314	1,612,657	75.1	914,208	457,104	25.5	2,190,356
Louisiana	1,049,009	524,310	36.1	11,507,992	2,148,774	32.5	1,761,038	775,592	28.6	2,272,042
Maine	2,606,505	1,298,831	61.9	1,476,101	738,050	32.6	227,424	113,711	1.9	160,661
Maryland	520,000	260,000	8.6	2,333,767	1,160,121	40.9	1,034,781	498,730	13.1	1,670,916
Massachusetts	1,581,901	790,948	7.1	2,862,339	1,431,167	16.4	771,127	383,820	11.4	2,200,354
Michigan	6,353,733	3,081,704	132.5	4,664,328	2,330,647	134.8	942,775	453,170	14.8	1,350,304
Minnesota	3,605,384	1,743,673	200.2	6,566,335	3,260,535	317.8	1,153,760	575,975	61.2	1,888,018
Mississippi	1,654,408	785,862	63.5	9,285,812	3,531,207	409.5	2,387,560	974,250	92.2	1,734,726
Missouri	3,932,508	1,890,869	113.9	2,595,248	1,287,136	59.2	5,011,716	2,272,236	194.0	2,753,696
Montana	1,666,619	936,796	83.6	703,632	395,460	16.4	312,764	165,665	9.6	4,315,758
Nebraska	2,993,074	1,496,429	299.7	5,525,012	2,754,035	415.7	4,056,160	1,394,528	263.6	1,450,619
Nevada	1,002,368	856,485	151.4	1,004,456	864,644	33.6	806,542	699,279	34.8	678,332
New Hampshire	831,761	437,187	21.5	437,424	217,752	3.3	93,802	46,900	1.4	1,141,725
New Jersey	793,715	388,600	6.3	2,786,016	1,390,128	19.8	895,990	447,165	9.6	2,254,697
New Mexico	1,966,562	1,198,902	220.1	1,481,057	1,003,464	68.0	1,022,421	580,234	57.3	295,370
New York	10,831,152	5,342,590	206.8	12,032,857	5,942,662	190.2	2,636,100	1,067,150	45.6	921,204
North Carolina	5,841,282	2,801,559	228.1	5,209,119	2,598,582	336.9	815,080	395,150	45.3	1,660,945
North Dakota	3,322,708	3,191,747	254.8	456,801	267,254	56.9	83,622	44,787	7.4	3,568,606
Ohio	5,627,144	2,787,529	68.7	9,580,762	4,762,200	93.0	1,789,340	891,514	19.2	5,487,881
Oklahoma	4,269,577	2,256,996	182.5	3,931,773	2,039,695	114.8	1,980,100	1,037,467	44.1	2,144,066
Oregon	2,874,047	1,693,690	106.8	1,225,473	740,297	62.6	917,078	559,320	50.1	1,427,148
Pennsylvania	6,439,721	3,198,425	111.5	7,493,866	3,739,756	83.7	3,411,363	1,686,038	34.8	2,319,486
Rhode Island	1,131,980	565,990	16.1	405,922	202,961	3.9	73,900	36,950	.6	1,028,924
South Carolina	3,470,957	1,524,748	204.8	3,977,180	1,789,376	112.2	480,641	223,700	18.0	1,241,355
South Dakota	1,807,402	985,571	215.0	4,411,146	2,439,470	400.3	772,680	427,270	91.9	2,563,496
Tennessee	2,627,760	1,313,880	95.0	4,319,134	2,159,567	102.2	1,291,960	645,480	28.3	3,872,020
Texas	9,845,961	4,873,446	666.3	9,682,145	4,649,822	441.1	5,851,109	2,845,867	246.8	4,405,759
Utah	1,111,437	785,540	114.7	1,869,040	1,329,150	65.4	182,350	128,000	6.4	575,283
Vermont	1,188,228	548,588	32.3	808,484	379,703	20.3	196,970	98,295	4.3	161,742
Virginia	5,218,691	2,607,165	181.5	3,342,173	1,669,251	104.4	697,992	346,891	14.9	571,864
Washington	3,897,164	2,032,732	91.8	2,535,423	1,329,759	23.2	337,657	177,400	3.1	677,739
West Virginia	1,322,093	874,029	43.6	1,637,996	995,455	44.9	850,210	421,255	22.7	1,995,928
Wisconsin	3,826,117	1,863,271	126.7	6,888,313	3,259,780	199.2	164,594	78,000	.2	1,379,283
Wyoming	2,430,882	1,489,487	286.3	244,882	145,791	20.3	343,970	212,540	51.0	534,495
District of Columbia										
Hawaii	517,508	258,624	9.9	1,132,210	557,165	17.4	17,980	6,630		1,202,757
Puerto Rico				1,193,969	594,805	24.7	324,050	160,960	6.3	478,610
TOTALS	154,490,584	80,969,973	6,859.5	198,662,145	97,386,077	6,082.8	59,061,194	28,864,560	2,090.6	90,171,545

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF NOVEMBER 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossing Protected by Signals or Otherwise	
Alabama	$ 252,184	$ 252,110	6		4	$ 555,574	$ 554,624	5	1		$ 402,505	$ 401,600	4		3	$ 777,185
Arizona						9,452	9,452									620,761
Arkansas	46,565	45,628	3			497,443	497,052	9			177,787	177,678	3			1,038,411
California	669,417	669,417	2	2		1,095,370	1,094,795	6	2		719,415	718,935	3			1,217,062
Colorado	30,465	27,839	1		1	61,728	61,728	1			120,759	116,999	1		18	1,081,746
Connecticut						18,930	12,665									831,825
Delaware	24,500	24,500			7						61,920	61,920	2		16	407,330
Florida						215,316	215,316	2			207,100	207,100	1			983,381
Georgia						96,720	96,720	2			305,230	305,230	4		1	2,015,437
Idaho	174,973	174,800	4			263,415	249,386	4								398,684
Illinois	286,500	286,500	2		1	1,618,825	1,618,825	8	2	9	1,040,690	1,040,690	7		66	2,258,628
Indiana	408,486	322,500	2	3		1,032,980	1,006,080	3	3	89	271,000	271,000	1	1		814,569
Iowa	951,920	900,897	9	1	14	265,413	249,900	3		5	47,124	44,200	1	2	2	1,372,564
Kansas	456,074	456,074	8		4	605,225	605,225	9			532,921	532,921	5		6	980,288
Kentucky	145,000	145,000	1			291,525	291,525	2	1	23	494,414	494,414	8	1	8	883,106
Louisiana						225,255	196,478	3			476,771	473,090	10			907,503
Maine	48,590	48,590	2			327,315	327,315	3	2		69,830	69,830	1		1	245,468
Maryland						64,586	64,586	1								962,247
Massachusetts	54,710	54,710		1		220,486	220,290	1	1		222,275	220,344		3		1,417,944
Michigan	893,783	887,372	8	1	16	622,336	622,336	6	1	29	80,690	80,690		1		1,694,787
Minnesota	40,218	40,218		1	1	760,185	759,864	3	5		18,297	18,297			3	1,626,755
Mississippi						465,800	465,800	6			250,000	250,000	1			868,351
Missouri	128,514	127,392	2	1		279,530	279,530	3			397,960	397,960	1	1		2,220,371
Montana	355,586	350,704	4			449,518	449,518	4			409,042	409,042	5			115,207
Nebraska	150,374	150,374	4			390,393	390,393	10			738,215	738,215	20		33	485,428
Nevada	97,187	97,187		2	2	55,074	55,074		1		231,055	231,055	2		4	75,325
New Hampshire	61,732	61,425		1		91,606	91,547	5		1	4,590	4,590			2	336,188
New Jersey	116,891	111,665	1	1		223,914	223,914	1	1	2						1,634,678
New Mexico	168,984	168,984	4	1		118,994	118,994	3								563,513
New York	141,400	141,400		1	2	2,447,252	2,438,751	9	8		284,420	189,030		2	1	4,000,748
North Carolina	73,550	73,550		1		917,500	884,800	7	5		147,030	147,030	1		11	1,339,014
North Dakota						822,138	822,138	4	2							757,083
Ohio						95,850	95,850	2			640,700	640,700	7			3,492,618
Oklahoma						17,343	17,343				250,305	250,305	1		55	2,013,861
Oregon	308,391	307,742	1	2		58,634	58,634				129,997	129,997	2			478,958
Pennsylvania	138,043	122,837	1			304,601	299,694	2			1,559,577	1,240,792	3			4,069,212
Rhode Island						335,019	335,019	1	2		123,785	123,785	1			34,946
South Carolina	20,930	20,930				164,359	109,393	3	2		335,820	335,820	12		40	1,022,959
South Dakota	99,940	99,940	1		8	241,918	241,918	2	2	1	89,610	89,610	4		3	888,632
Tennessee						14,381	14,381				320,910	320,910	1	2	5	1,556,742
Texas	34,033	33,377	2			1,219,529	1,218,452	12	5		425,695	379,260	5			3,712,027
Utah	97,174	97,174	2		1	47,359	47,359	2			5,319	5,319			3	423,211
Vermont	202,882	197,882	6	1		43,274	43,274		1	6	18,330	18,330			6	220,966
Virginia	248,306	248,306	12	1		241,018	241,018	3	1		397,338	397,338	6	1	8	909,368
Washington	149,718	147,618	1	2	2	673,771	673,771	8	2		128,216	128,216	1		9	434,195
West Virginia	120,100	119,080	1			315,009	315,009	1			175,870	158,870	4		6	731,139
Wisconsin	162,493	162,493	2			1,246,886	1,160,026	12								1,150,990
Wyoming	156,370	156,370		2	1	10,150	10,150				24,840	24,840			10	485,370
District of Columbia											34,262	34,262		1		291,169
Hawaii						193,200	193,200	3	1							300,550
Puerto Rico						214,569	213,370	4								516,930
TOTALS	7,515,983	7,332,585	92	25	64	20,546,698	20,262,482	178	51	165	12,371,614	11,880,214	128	15	320	57,665,430

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF NOVEMBER 30, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 73,700	$ 36,850	10.7	$ 565,005	$ 281,050	33.3	$ 262,200	$ 130,350	5.6	$ 604,722
Arizona	281,361	187,459	18.6	190,653	116,491	17.5	7,440	5,357		347,175
Arkansas				84,417	77,799	8.0	267,422	266,718	25.6	519,591
California	672,360	388,635	60.3	1,188,992	641,483	65.3	598,608	324,358	11.9	493,236
Colorado	551,856	306,038	37.3	574,530	299,145	30.1	75,210	39,480	3.0	276,905
Connecticut	48,450	24,205	1.0	62,584	31,132	.5	4,700	2,310		260,631
Delaware				49,590	24,795	10.9	69,994	34,512	11.9	187,568
Florida				112,922	56,461	.6	290,182	145,050	5.3	473,341
Georgia	226,099	109,101	29.4	558,626	279,313	66.5	239,540	119,770	36.0	750,538
Idaho	412,249	180,843	44.6	104,258	51,017	12.7	128,534	76,572	2.6	165,939
Illinois	985,586	492,025	83.2	1,812,092	852,046	117.2	457,700	220,350	35.0	463,730
Indiana	370,100	144,250	42.9	882,500	420,450	83.8	733,248	354,302	62.9	323,436
Iowa										1,298,449
Kansas	53,690	26,845	8.6	180,426	90,212	7.7	141,936	70,968	40.1	1,130,946
Kentucky	656,862	196,489	82.4	830,684	230,171	46.8	704,551	205,148	68.1	186,987
Louisiana				385,320	175,495	34.3	753,262	321,350	59.5	227,291
Maine	352,800	176,400	22.7	265,826	127,796	13.0	18,700	9,350	1.2	15,027
Maryland				45,164	15,787	5.8	308,800	110,355	18.4	286,334
Massachusetts				64,760	32,380	1.4	292,333	145,470	5.8	468,550
Michigan	129,761	63,381	17.2	678,004	339,002	51.0	546,350	273,175	24.0	859,981
Minnesota	217,978	101,203	30.8	484,234	237,735	50.0	421,664	210,832	24.0	829,810
Mississippi				299,000	149,500	23.8	90,000	45,000	8.4	694,427
Missouri	271,479	132,333	35.2	187,360	93,290	15.7	612,860	288,465	109.8	551,584
Montana				13,983	7,865					1,027,170
Nebraska	410,992	204,652	72.2	380,650	187,245	59.9	484,188	234,873	83.7	370,446
Nevada	420,554	361,693	68.8	74,601	61,643	10.2	43,720	26,016	4.8	45,000
New Hampshire	201,355	99,875	4.5	47,226	23,583	1.5	81,281	39,806	2.4	83,611
New Jersey	71,490	35,745	1.8	202,630	99,470	3.2	48,780	17,500	.1	520,058
New Mexico	516,802	315,195	30.7	436,367	207,627	18.3	337,887	192,174	28.8	91,776
New York	1,990,034	991,719	145.6	2,041,800	1,020,900	108.9	178,400	89,200	11.4	260,675
North Carolina	461,268	230,634	48.3	902,084	451,020	80.4	177,160	75,890	14.3	324,311
North Dakota	33,860	18,135	5.5	119,670	64,092	29.3	115,886	62,065	8.5	642,960
Ohio	156,560	78,280	3.8	27,840	13,920		349,200	174,050	19.2	1,565,391
Oklahoma	21,800	11,598	1.1	426,228	225,671	37.2	529,440	258,520	30.9	689,126
Oregon	453,626	266,110	58.5	38,310	23,422	2.1	21,144	12,900	2.9	452,701
Pennsylvania	1,335,720	644,619	99.7	1,605,145	780,641	85.5	824,754	408,877	49.3	282,547
Rhode Island	66,840	33,420	3.5	162,675	81,314	4.8	74,070	37,035	.9	36,122
South Carolina	274,331	123,372	31.0	874,399	363,769	93.8	251,400	94,300	21.3	97,239
South Dakota				11,300	6,250					816,436
Tennessee	78,900	39,450	2.3	661,226	257,713	35.8	195,460	91,260	7.2	671,067
Texas	1,561,835	730,571	247.3	1,958,249	853,904	204.1	868,757	392,535	98.4	1,147,759
Utah	403,316	222,870	40.1	222,529	118,625	15.2	53,420	22,360	8.1	186,515
Vermont	240,650	108,150	13.8	90,306	45,153	4.0	43,300	20,500	.5	35,793
Virginia	360,647	178,108	43.8	367,320	383,720	60.4	26,300	13,150	1.9	317,407
Washington	428,077	223,948	45.0	334,946	176,178	26.6	411,332	216,400	29.2	114,030
West Virginia	124,300	62,150	9.5	228,700	114,350	18.1				373,174
Wisconsin	261,201	120,812	6.1	903,452	440,030	47.0	80,514	40,090	2.7	611,296
Wyoming	431,531	266,644	59.0	301,302	186,179	11.9	136,438	84,301	10.4	48,069
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico	39,770	19,885	1.4	232,480	115,765	13.8	96,700	47,210	4.9	64,015
TOTALS	15,650,290	7,953,692	1,568.2	22,828,665	10,961,324	1,670.3	12,454,765	6,050,254	1,000.9	22,509,642

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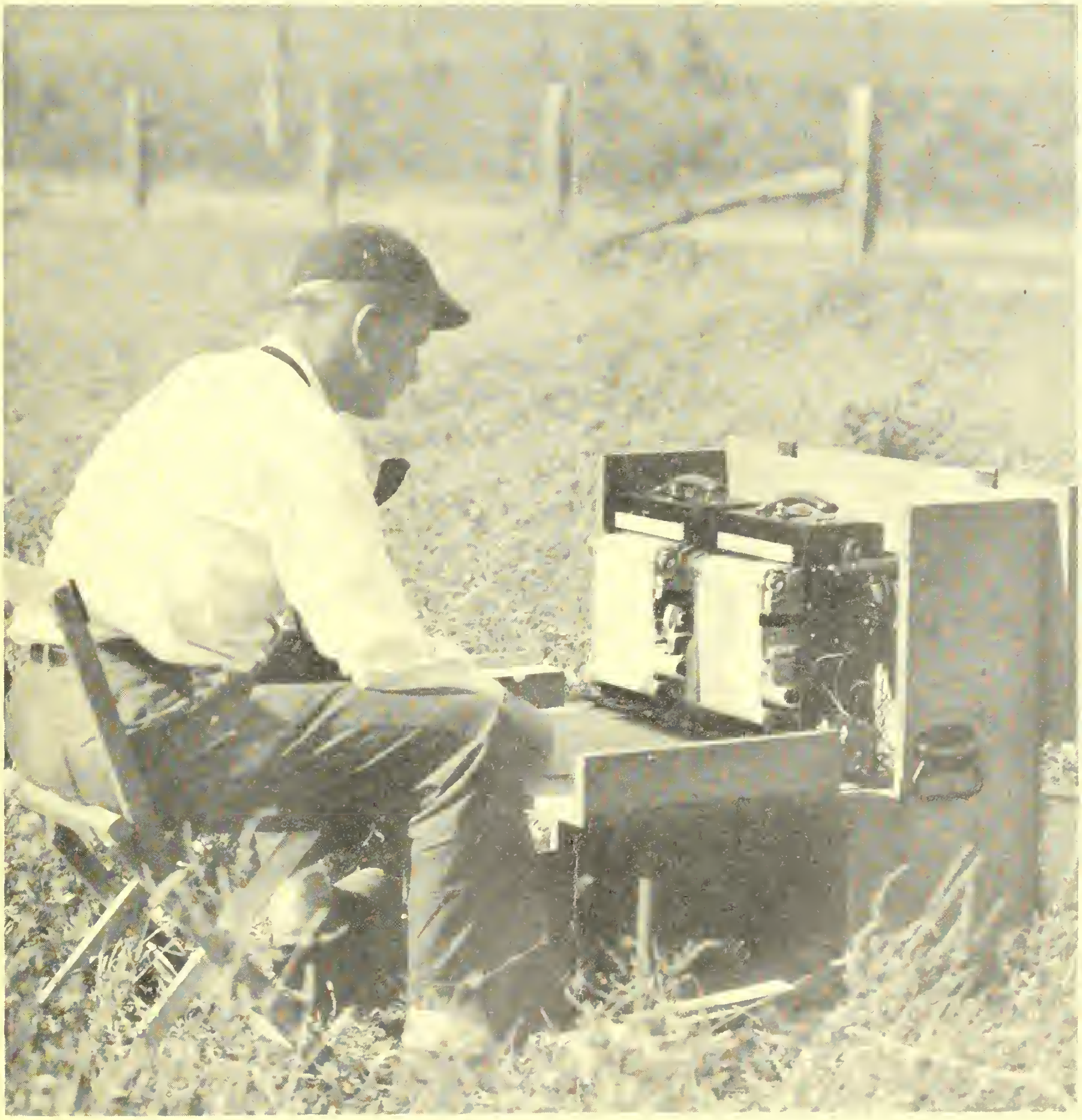
PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 11 JANUARY 1939



GRAPHIC RECORDERS USED IN FIELD STUDY OF VEHICLE PASSING PRACTICES

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PUBLIC ROADS ▸ ▸ ▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 11 January 1939

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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PROCEDURE EMPLOYED IN ANALYZING PASSING PRACTICES OF MOTOR VEHICLES

Reported by E. H. HOLMES, Highway Engineer-Economist, Bureau of Public Roads

THROUGHOUT the entire course of highway development in this country, there has been constant and painstaking research; in fact, it may be said with practical certainty that only by intensive research has such rapid development been possible. The efforts of the many agencies studying their individual problems have been coordinated, and the results of their work integrated, evaluated, and made available to the benefit of all engaged in such activity. But, through force of necessity, the engineers have devoted by far the greater portion of their efforts to the development of a satisfactory structure. First, it was necessary to determine the most suitable combinations of available materials to provide a structure physically capable of supporting the loads imposed by the traffic using it, over a base frequently changing in its supporting power through changes in climatic conditions.

With advances in knowledge of the use of available materials, attention was directed to the refinement of these materials and to the development of others to provide even stronger and more durable surfaces. With the development in the uses of materials came improvement in the processes of their combination, and of their application and manipulation in the field to decrease the cost of those most generally used and to make available for general use those previously impracticable.

During all this development in technique and theory, little attention was given to the alinement of the highway other than to adhere to standards which all too frequently were quite arbitrarily defined. Grades generally were limited to a certain percent and curves to a fixed degree, but too often local conditions were accepted as adequate reasons for disregarding the general limitations. From the beginning, economy of construction was a major factor in the choice of grade and alinement. Calling upon the experience of years of railroad practice, highway engineers balanced cut and fill, and introduced waste and borrow as necessary to provide an alinement cheapest to follow within the limits of the standards adopted.

Consideration of such factors as sight distance and superelevation, for example, was, and generally still is, on the basis of single vehicles. Limits were based on the distance required for a vehicle to stop from a given speed or on the superelevation required for a vehicle to negotiate given curves at given speeds. It is true that vehicles have been studied in quantity, as well as individually, but such studies have been confined to their total number and their classification as a justification for certain types of surfacing, or to their dimensions and weights for information on the required structural strength of the surfaces. Even up to the present time, there has been no concerted, intelligently directed effort to determine the effectiveness of the highway in performing one of its most important functions, that of permitting the safe and expeditious movement of traffic.

Railroad design did not contemplate the promiscuous passing of vehicles moving in the same direction. Fast-moving trains pass slower freights or passenger trains at scheduled times and at predetermined places, not at the particular time chosen as advisable or expedient by the engineer, and passing sidings can be constructed where the topography and other features indicate they are the most economical. But there is no justification for building, for example, a three-lane road, perfectly designed as to economy of construction, if its alinement is such that traffic must be restricted to two lanes on frequently recurring hillcrests or curves. In providing opportunity for passing only where the topography is especially suitable, the road does not fulfill the demands of the traffic which justified it. In short, past research has been largely directed toward the economy of construction, and there is ample evidence that in too many instances concentration on this factor has been at the expense of the economy of alinement, as measured by the effectiveness of the completed highway as a medium of economical transportation.

FIELD STUDY OF PASSING PRACTICES NECESSARY

In the future, research in materials and in the methods of their placement must be continued, but much more emphasis must be placed on this necessary analysis of the effectiveness of the highways built. In fact, it is likely that highway research of the future will be concentrated on the design of alinement, grades, and widths of particular routes, and on the relationship of entire highway systems to the needs of the public, as well as upon the design of the surfaces themselves. In this future analysis work, there must be constant improvement of existing methods of traffic analysis, and the development of entirely new methods for studying the new problems which will be presented.

Coincident with this development of methods, there will be required the design and development of instrumentation to make possible the gathering in the field of the data required by these various analysis methods. In the future, it will not be sufficient to confine the analysis of the movement of traffic to the movement of individual units of the traffic stream. A sufficient analysis must be broad enough in its scope to include all vehicles using the highway. It may be considered a problem of dynamics in which it is necessary not only to study the movement of the individual units of the traffic stream, but also to determine how the movement of these units is affected by the external forces within and without the stream itself.

Of course, in the actual analysis of this whole problem, the approach must be by small, independent, yet related investigations. Two separate studies bearing on the problem are now being conducted by the Bureau of Public Roads.[1] A study of the speed and spacing of vehicles has enabled conclusions to be drawn as to the normal habits of drivers under various traffic conditions and to develop indices to measure when, in their normal driving over present highways, drivers become aware of traffic congestion.

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[1] Preliminary reports of these studies will be published in the February 1939 issue of PUBLIC ROADS.

In another study, analysis is being made of the performance of motortrucks. Since trucks constitute a known portion of highway traffic, results of this study can be utilized in conjunction with the findings of studies of the normal habits of all drivers. It will be possible to determine not only reasonable performance factors for these vehicles, but also the effect slow-moving vehicles have when they are found in varying percentages in the traffic stream. Still another study deals with the transverse placement of vehicles on the highway. By means of electrical instruments the position of each vehicle is automatically recorded as it passes a given point, while speed meters, now in the process of development, will simultaneously record the speeds. Only by a coordination of such studies can standards be evolved that will result in the greatest latitude in the design of the highway and of the vehicle consistent with the free movement of traffic.

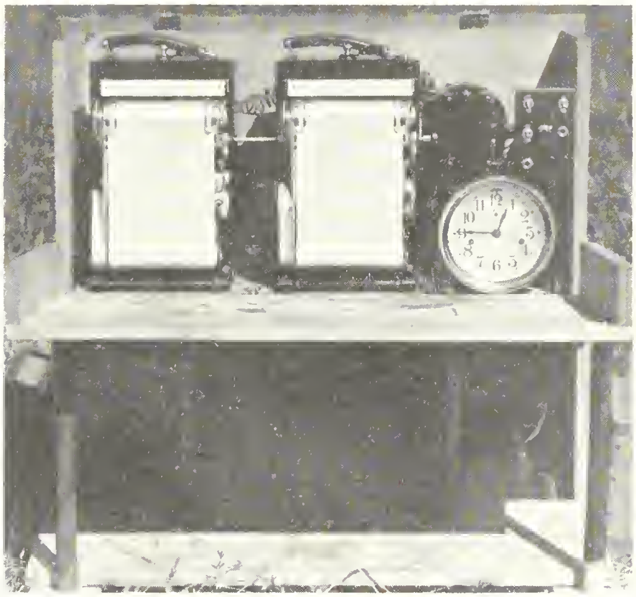


Figure 1.—One Set of Graphic Recorders, and Master Clock Used to Synchronize all Six Recorders.

Still another study in this series of closely related investigations is an analysis of passing practices. Studies of passing distances, both by analytical and experimental methods, have frequently been undertaken, and their results have been useful. Yet it is believed that none of these studies, generally confined to the mechanics of passings, has been sufficiently exhaustive in its nature to answer all the questions the highway designer should ask. It is not difficult to determine the distance required for one vehicle to pass another under various conditions of speeds of the two vehicles, but the results are of little value unless the frequency of concurrence of passings at various speeds is also known. Similarly, it is of little value to know the frequency with which such simple passings occur, if the majority of passings are accomplished not singly, but in groups. A comprehensive analysis of passing practices must, therefore, be built around the following specifications as an absolute minimum:

1. The study must be conducted in the field.
2. The study must include only the normal traffic. Test drivers or test vehicles cannot be employed in determining normal passing practices.
3. All the units of the traffic stream must be observed, and their progress recorded continuously.
4. The section over which the study is conducted must be of sufficient length to permit the completion of any normal passing maneuver.
5. Since observation of normal driving practice is required, the work must be so distributed geographically that all differences in driving habits are included.

In the development of a method satisfying these requirements, a half-mile section of roadway was decided upon as a suitable length to comply with requirement 4. Whether it was too short or whether it involved unnecessary work in being too long can only be determined by analysis of the field data. Indications are, however, that this length was ample. It then became necessary to find a means of recording the progress of all vehicles as they traversed the section. Many methods were considered and discarded without trial in the field. Others, including various photographic means, were investigated in the field, and of these the method chosen involved an adaption of a type of equipment previously employed for highway-capacity and other studies.

DETECTORS SPACED AT 50-FOOT INTERVALS ALONG HALF-MILE SECTION OF HIGHWAY

The method required that detectors be placed in each traffic lane at 50-foot intervals. Each detector was connected by a cable to an individual pen of a graphic time recorder, which recorded on a strip-chart the time at which each passing vehicle actuated successive detectors in the particular lane in which it was traveling. Since this strip-chart moves at a constant speed, to determine the speed of any vehicle over any 50-foot section required only that the distance on the chart between the marks made by successive pens be scaled, and that this measurement be converted to speed. The precision of measurement of vehicle speed thus is a function of the chart speed. In these studies, a chart speed of 45 inches per minute permitted the determination of the vehicle speed to within 2 miles per hour at 60-mile-per-hour speeds and with correspondingly greater precision at slower speeds. Over 100-foot sections, of course, the possible error is but half as great.

Observation over a half-mile section required 108 detectors, 54 for each lane of the two-lane roads, and, correspondingly, time recording apparatus with a similar number of pens. Fortunately, recording equipment ideally adapted to this purpose was available commercially, and experience with these particular instruments in previous studies by the Bureau indicated that they would dependably and accurately record all the movements required. The recording apparatus, as it was assembled in this study, consisted of a number of 20-pen time recorders, normally driven by clockwork but which could be coupled together in pairs to form 40-pen units, the entire units being driven by external electric motors to provide the fast chart feed required for the work. A complete recording unit is shown in figure 1.

The entire half-mile of highway was divided into three independent sections. Within each section, the detectors were connected to a single 40-pen recording unit, thus making it desirable that the recording units be located at three separate points. While the two charts in a single unit were constantly synchronized by the mechanical coupling between the two recorders, it was necessary to synchronize the three units by means of a single master clock in series with a pen in each of the six recorders. This clock automatically closed a circuit and actuated the six pens at 10-second intervals, while the time of day was recorded in code, by means of a telegraph key in the same circuit, at intervals of 2 or 3 minutes.

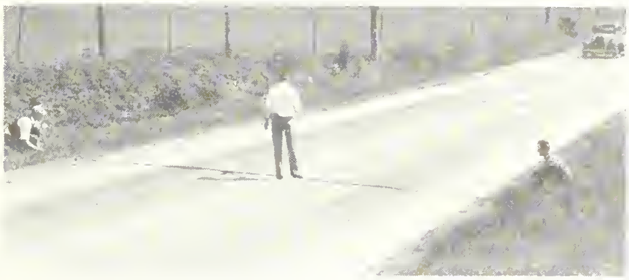


Figure 2.—Detector Tube Being Installed. The Tube is Centered and the Ends Fastened by Spikes.

The most difficult feature of the instrumentation was the provision of satisfactory detectors. An exhaustive study of available detectors revealed that none could be relied upon even for counting traffic, to say nothing of satisfying the more rigid requirements of this particular study. Cheapness in original cost, inasmuch as over 100 were required, was an important item, but ease of handling, both in installation and in transit, was equally essential.

As a result of concentrated efforts in the development of such a detector, a design employing an air switch and a rubber tube was finally adopted. The air switch itself was constructed from an automobile oil pressure unit by removing the original metal diaphragm and inserting in its place a sensitive diaphragm of rubber. To this revamped air switch was connected a rubber tube about three-eighths of an inch in external diameter to be laid across the pavement. Even the selection of a suitable tube involved considerable research since durability under traffic, resiliency, and a certain wall stiffness proved to be essential factors. Finally a tube composed of 95 percent pure gum rubber, liberally treated inside with talc, was adopted. As a vehicle passed over this tube, the air displaced by the wheels actuated the diaphragm and closed a circuit leading to the time recorders. The design of the complete detecting unit is described in detail on pages 214–216.

As the detector was adapted to use in the passing studies, two air switches were attached to each tube, one at each end, and holes were punched through the walls of the tube at its midpoint. These holes effectively prevented the air impulses from passing across the midpoint and each air switch could, therefore, be actuated only by the vehicles in the traffic lane nearest the switch. Thus a single tube and two air switches served as two detectors, and permitted positive identification of the lateral position of all vehicles with respect to the lane in which they traveled. Figure 2 shows the rubber tube being centered on the road during installation.

An interesting feature of the detector was the means by which the electrical circuits were conducted from the air switches to the recorders. There were, of course, two leads from each air switch. The negative leads from the switches on the side of the road opposite the recording equipment were connected to a single wire running the length of the section and serving as a common return. The positive lead from each of these switches was returned by a wire installed inside the tube to the side of the road near the recording equipment, and was there incorporated in the cable leading to the time recorders. The entire unit, including the tube and the two air switches enclosed in ordinary tin cans for protection from the weather, cost less than $4, and could be wound into a small coil when not in use. When installed at 50-foot intervals on a concrete road the tubes resembled expansion joints, and were barely visible on bituminous surfaces.

POSITION AND SPEED OF EACH VEHICLE RECORDED

Finally, to connect the road switches to the time recorders, some 50,000 feet of wire was made up into six cables, one cable to run in each direction from each recording unit. To the detectors at the ends of the section ran three wires, two for the positive leads and a common return for the negative lead to the switch on the near side of the road. While the common return was tapped at each detector, two wires for the positive leads were added to the cable at 50-foot intervals until it consisted of 19 wires for the road switch circuits as it reached the recording equipment. The recording units were connected by four more wires that were incorporated in the cable. Two of these wires were for the circuits for the synchronizing clock, and two were for the telephones used in communication between the three units. The common returns on the two sides of the road were connected by a wire laid across the pavement. The single return wire mentioned previously, with the leads accurately spaced at 50-foot intervals, was normally laid first, pulled taut and spiked at both ends. By using these leads in spacing the detectors, it was not necessary to measure the sections before or during installation.

The installation of the equipment is the most laborious feature of the field work, a minimum of 4 hours being required for six men to install all the equipment and prepare the recording units for operation. Once the installation is complete, the operation of the apparatus is a simple matter. With an attendant at each recording unit, all six recorders are started simultaneously. The charts, supplied in 100-foot rolls, last about 25 minutes at the chart speed utilized in this study. After one set of charts is exhausted, new ones are inserted in all the recorders and again the units are started simultaneously, the change-over normally requiring about 3 minutes.

During the time the charts are running through the recorders, the attendants record, by means of a telegraph key in circuit with a special pen, the passage of trucks or busses. By thus classifying passing vehicles, it will be possible to determine whether the type of vehicle is in itself a factor in passing practice.

During the preliminary work conducted during the fall months of 1938, it was customary to install the equipment on Saturday morning, and to operate it that afternoon and most of the daylight hours on Sunday. The equipment, with the exception of the detectors, was left in place over Saturday night. By conducting the studies in this way, at four different locations in the vicinity of Washington, D. C., one of which is shown in figure 3, information has been accumulated on approximately 2,000 passings. These data are now being analyzed in the office. It is expected that analysis of a major part of the field data already

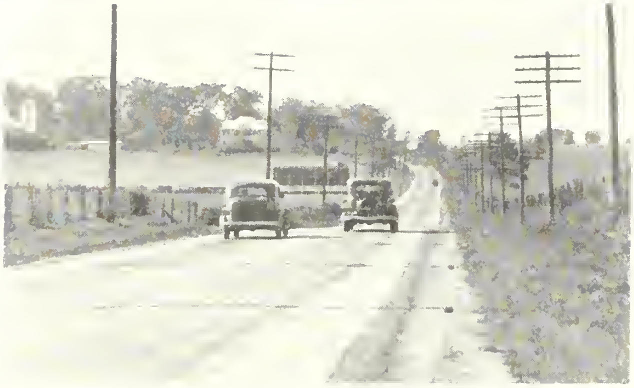


FIGURE 3.—DETECTORS IN PLACE ON A SECTION OF ROAD ON WHICH PASSING PRACTICES WERE OBSERVED.

collected will be completed before field work is resumed in the summer months of 1939.

The records of simple passings as they appear on the charts are not difficult to interpret, although passings involving several vehicles are naturally more complicated. The pens in the two recorders of each unit were so connected with the road switches that, as a vehicle travels in its own lane through the section, its progress is recorded on successive pens on one machine. As a vehicle traverses the section in the opposite direction in its own lane, it actuates successive pens in the other of the two recorders. If, however, a vehicle moves in the left lane, it actuates the pens in the recorder connected to the switches in that lane, but in the reverse order, while if it straddles the center line it actuates similarly numbered pens in the two recorders simultaneously.

Thus in a simple passing the two vehicles engaged in the maneuver are first recorded on the chart for the right lane. As the second vehicle draws left to begin passing, its progress is charted on both recorders while it straddles the center line. Soon it moves completely into the left lane and its progress is noted only on the recorder for that lane, where the pens are actuated in reverse order. After having passed the first vehicle, the passing vehicle is recorded again as it straddles the center line in returning to its own lane; and finally, as it draws away from the vehicle just passed, it again is recorded only in the right lane. Meantime, the passed vehicle has been maintaining its course along the right lane, and the effect of the passing on this vehicle may readily be observed by determining whether it accelerated, decelerated, or continued at the same speed while the other vehicle passed and drew away.

STUDY DETERMINES ACTUAL DRIVING PRACTICES

Although interpretation of the field data is not difficult, its transcription and summarization is extremely laborious. Analysis of two or three passings per clerk per day is all that is possible in view of the completeness of the information obtained. The figures in table 1 were summarized from the transcribed data from two simple passings.

TABLE 1.—*Sample data for two simple vehicle passings*

Item	Passing	
	No. 1	No. 2
Speed of passed vehicle before the passing .... miles per hour..	26.2	26.2
Speed of passing vehicle before the passing .... do....	32.0	31.0
Maximum speed of passed vehicle during the passing .... do....	29.1	27.3
Maximum speed of passing vehicle during the passing .... do....	37.9	37.9
Speed maintained by passed vehicle after the passing .... do....	29.6	23.5
Speed maintained by passing vehicle after the passing .... do....	42.6	37.9
Speed difference before the passing .... do....	5.8	4.8
Maximum speed difference during the passing .... do....	8.8	10.6
Speed difference after the passing .... do....	13.0	14.4
Distance passing vehicle straddled center line in beginning passing .... feet..	100	50
Distance passing vehicle traveled entirely in left lane .... do....	400	200
Distance passing vehicle straddled center line in completing passing .... feet..	350	100
Distance passing vehicle encroached in left lane .... do....	850	350
Time passing vehicle encroached in left lane .... seconds..	15.4	7.0
Speed of approaching vehicle .... miles per hour..	(1)	34.1
Distance between passing vehicle and vehicle approaching from the other direction immediately before passing .... feet..	(1)	792
Distance approaching vehicle traveled during passing .... do....	(1)	349
Clearance between passing and approaching vehicles at completion of passing .... feet..	(1)	93

[1] None involved.

Although some of these data will not be required in the immediate analysis, summarizing and tabulating the material in detail is essential to permit any future analyses that may be required. It is obvious that in each simple passing there are four major variables. Three of them—the speed of the passed vehicle, the speed of the passing vehicle, and the speed of the approaching vehicle, if any,—are independent variables while the fourth, the dependent variable, is, of course, the distance required for the completion of the maneuver. In addition to these major variables, there are, particularly in multiple passings, a large number of minor variables, many of which will be significant in an analysis of the entire problem.

With respect to the relative importance of the various phases of the analysis, it should be apparent that the times and distances involved in individual passings are of minor importance. It is true that in the design of highways, distances required to pass under the various conditions normally encountered must be known in order to provide the required sight distances. These distances are, again, those needed in the design of the structure and add but little to our knowledge of the effectiveness of the highway in providing for the free movement of traffic.

Toward the end of interpreting the effectiveness of the highways as a medium of transportation, however, it is likely that information on the time required to pass, rather than the distance in which the passing is completed, will be of major significance, for it becomes increasingly evident that the time spacing of vehicles may follow some rather definite laws. Accordingly, a correlation between the time required to pass and the time spaces normally available in the opposing traffic lane will yield almost positive information with respect to highway capacity.

But it is of far greater importance, in the analysis of the entire problem of vehicular movement, to understand, rather than the elements of time and space involved in these individual or multiple passings, the actual behavior of the driving public. It is necessary to examine closely how passings are accomplished and in what number they may be expected, whether there is a preponderance of single passings, as mentioned before, or whether the majority of passings are accomplished by groups of vehicles passing other groups, and whether the alinement of the highway itself perhaps has a greater influence on the passing practices than do the psychology and desires of the individual drivers found in normal highway traffic. Studies that have already been completed along these and other lines have indicated that this driver psychology plays an important part in our traffic problem.

(Continued on page 221)

A SIMPLE PORTABLE AUTOMATIC TRAFFIC COUNTER

Reported by R. E. CRAIG, Junior Highway Engineer, and S. E. REYMER, Chief Scientific Aide, Bureau of Public Roads

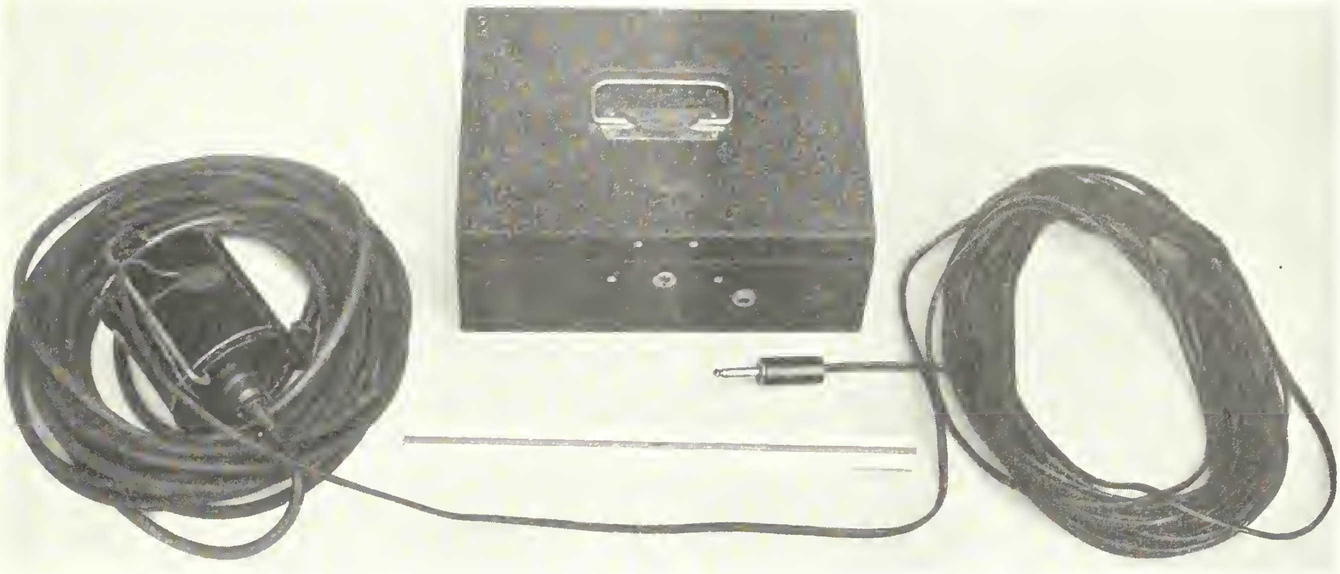


FIGURE 1.—PORTABLE AUTOMATIC TRAFFIC COUNTER, CONSISTING OF: LEFT, ROAD SWITCH AND RUBBER TUBE; MIDDLE, BOX HOUSING COUNTING UNIT AND BATTERIES; AND RIGHT, CABLE CONNECTING PNEUMATIC DETECTOR TO COUNTER.

MOST of the States now engaged in the highway planning surveys have completed the major part of the field work in connection with the traffic surveys. These traffic surveys have necessarily been extremely detailed in their nature. Not only has the geographical coverage been extensive but at many key points the traffic patterns have been intensively studied using data obtained by regularly repeated manual counts and by permanently installed automatic traffic recorders. Traffic has been classified by vehicle types on practically all roads throughout the various States and, on the more important highways, volume and classification counts have been supplemented by studies of commodity movements, of the origin and destination of both passenger and commercial traffic, of the weights and dimensions of commercial vehicles, and many other special localized studies.

This information, now in the process of tabulation and analysis will provide knowledge necessary to a determination of present traffic requirements. Since appreciable increases in traffic volumes are a future probability, it is essential that traffic counts be continued as a measure of future traffic requirements. It is believed that, with the exception of the material changes either in the volume or classification of the traffic in small areas because of extensive construction projects or significant economic changes, the general trends of traffic flow will remain rather uniform throughout a given area. Therefore, it should be possible to measure the increases or decreases of the total traffic volume by means of a relatively few counts properly distributed as to time and location.

Since in a determination of changes in the general level of traffic flow, vehicle classification and other detailed information become relatively less important, it should be possible to collect all the necessary information readily and cheaply by means of automatic traffic counters, should such machines be inexpensive and easily portable. The idea of employing portable counters, either of the recording or nonrecording type, is not new. The need for such counters has long been felt, and a number of agencies have experimented, some at the expense of considerable time and money, with the object of producing a suitable portable counter.

Simple accumulating type counter.—When equipment was first being developed by the Bureau of Public Roads for use in the study of motor-vehicle passing practices, discussed in the preceding report, no satisfactory portable traffic counters were available, principally because a suitable means of detecting the passing vehicles had not yet been developed. The success with which the pneumatic detector, used in these passing studies, actuated the recording elements in the time recorders, led the Bureau to believe that these same or similar detectors might be adapted to a properly designed counting mechanism to permit the construction of a low-priced, self-contained traffic counter. Some experimenting was necessary in order to determine the proper electrical constants to be included in a satisfactory counting circuit but, in the end, very few changes in the detector were required.

Throughout the development work, the primary requirements were simplicity, ruggedness, portability, low cost, ease of installation, and ability to make an accurate count of vehicles traveling at high speeds.

The traffic counter as now developed, shown in figure 1, consists of an electrically operated counting unit connected by a cable to a pneumatically operated detector.

The counting unit is comprised of an electromagnetic counter of the type employed as telephone call registers, an intermediate relay, the necessary batteries for operating the counter and relay, and other circuit compo-

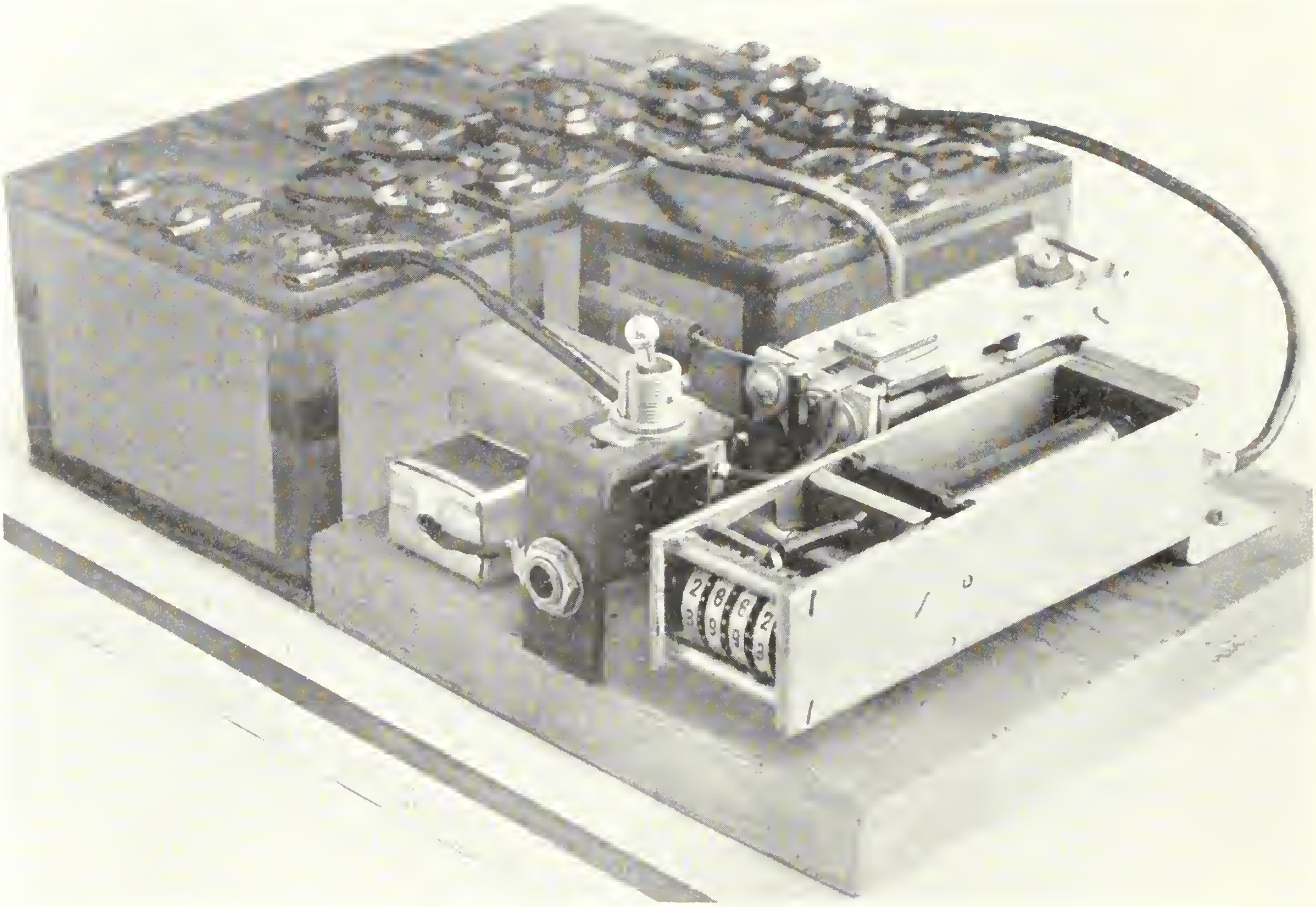


Figure 2.—The Counting Unit, Which Consists of an Electromagnetic Counter, an Intermediate Relay, Batteries for Operating the Counter and Relay, and Other Circuit Components.

nents. This apparatus is housed in a metal box provided with a hinged lid, lock, and carrying handle. The counter is mounted adjacent to a small window in the front side of the box, permitting it to be read from the exterior.

Figure 2 is an interior view of the counting unit, showing the arrangement of its component parts.

OPERATION OF ROAD SWITCH DESCRIBED IN DETAIL

Figure 3, top, shows the assembled pneumatic detector. The assembly consists of the road switch mounted on the inside of the lid of an ordinary tin container. As shown in the figure, the lid is held securely in place by a wire clamp that rides in a metallic guide fastened to the bottom of the tin container with a single brass screw that is also used to hold a spring clip. One terminal of the cable connecting the counting unit to the detector is connected to the spring clip, the other terminal of the cable is fastened, by means of a solderless connector, to a short length of insulated wire brought through a hole provided in the lid. Additional insulation is used where the wire goes through the lid. A one-fourth-inch iron washer and lock-nut are used to mount the road switch on the lid. Rubber tubing of the desired length is connected to the road switch by means of a one-eighth-inch brass nipple and reducer. The rubber tubing is cemented to the brass nipple with rubber cement.

Figure 3, bottom, shows the component parts of the road switch, which is a modified automobile oil pressure gage. In order to render the road switch sufficiently sensitive, a thin rubber diaphragm was substituted for the copper diaphragm with which it was originally equipped. The cover A is removed from the unit by turning off the crimped edge in a lathe. Only sufficient metal is cut away to permit the cover to be removed. The cover can be made to snap back onto the base D by slightly crimping opposite places on the rim of the cover A. Frame B, which carries the contacting elements, is removed from the base D by opening the three small ears. A seven-eighths-inch diameter hole is cut in the metallic diaphragm of the base D. The diaphragm C consists of a piece of thin sheet rubber sandwiched between and cemented to two paper disks. Diaphragm C is then cemented to base D with a rubber cement used for metallic surfaces. The one-eighth-inch brass reducer and nipple are marked E and F, respectively. The contacting elements on frame B were originally flexibly mounted with respect to their actuating elements. It was found necessary to secure them rigidly to the actuating elements as shown in figure 4 in order to eliminate vibration and consequent erroneous counts.

A wiring diagram of the traffic counter is shown in figure 5, the operation being as follows: With plug P inserted in the jack J, and battery switch SW in the "On" position, vehicles passing over the pneumatic detector cause contacts K_2 to close momentarily. Relay L_1 is then operated by the battery B closing contacts K_1 which in turn operate the counter L_2 from a higher voltage tap on battery B. Upon the closure of contacts K_2, the condenser C_1 is charged substantially

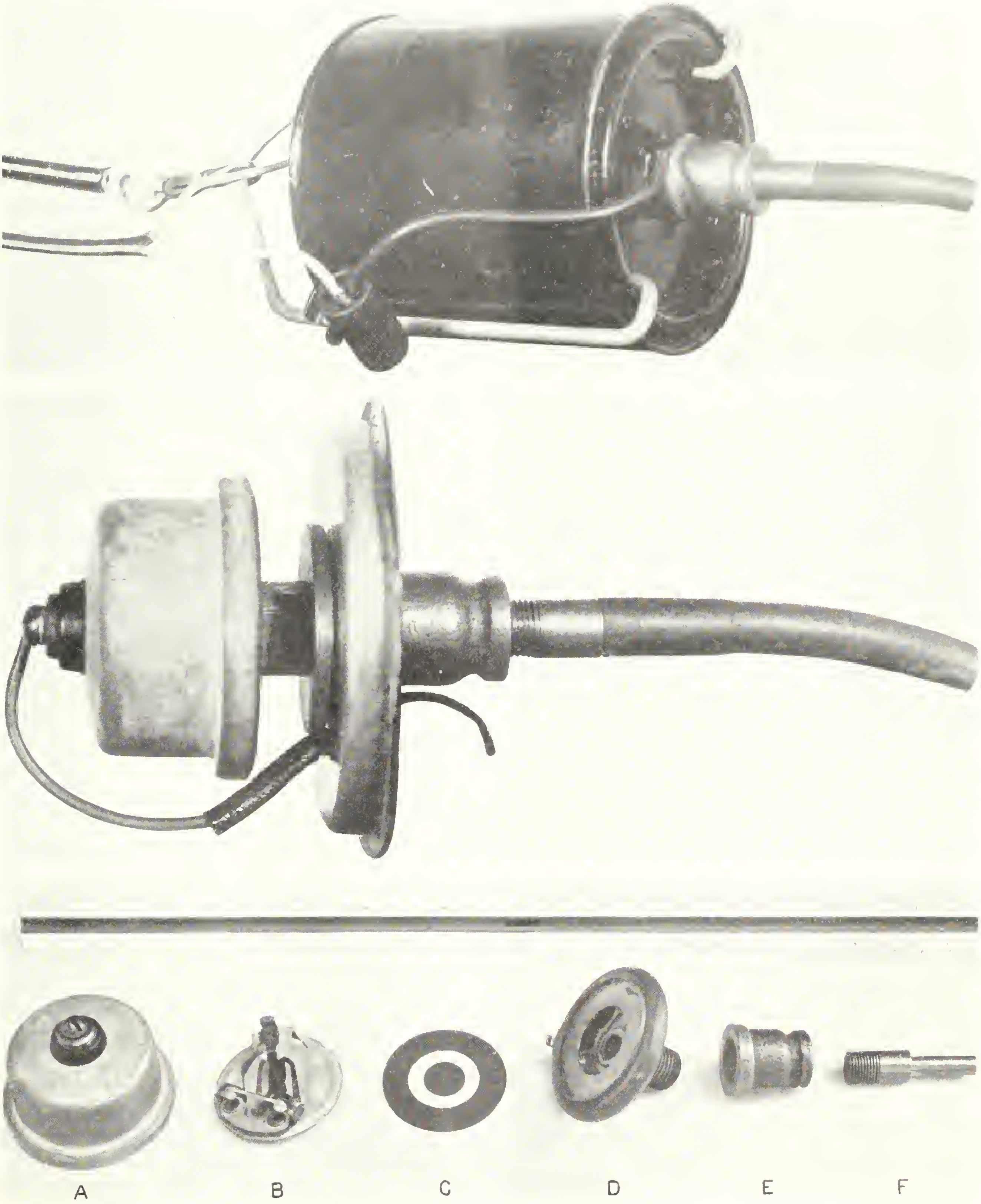


FIGURE 3.—TOP, THE ASSEMBLED ROAD SWITCH. CENTER, ROAD SWITCH WITH CAN AND FASTENINGS REMOVED. BOTTOM, THE COMPONENT PARTS OF THE ROAD SWITCH.

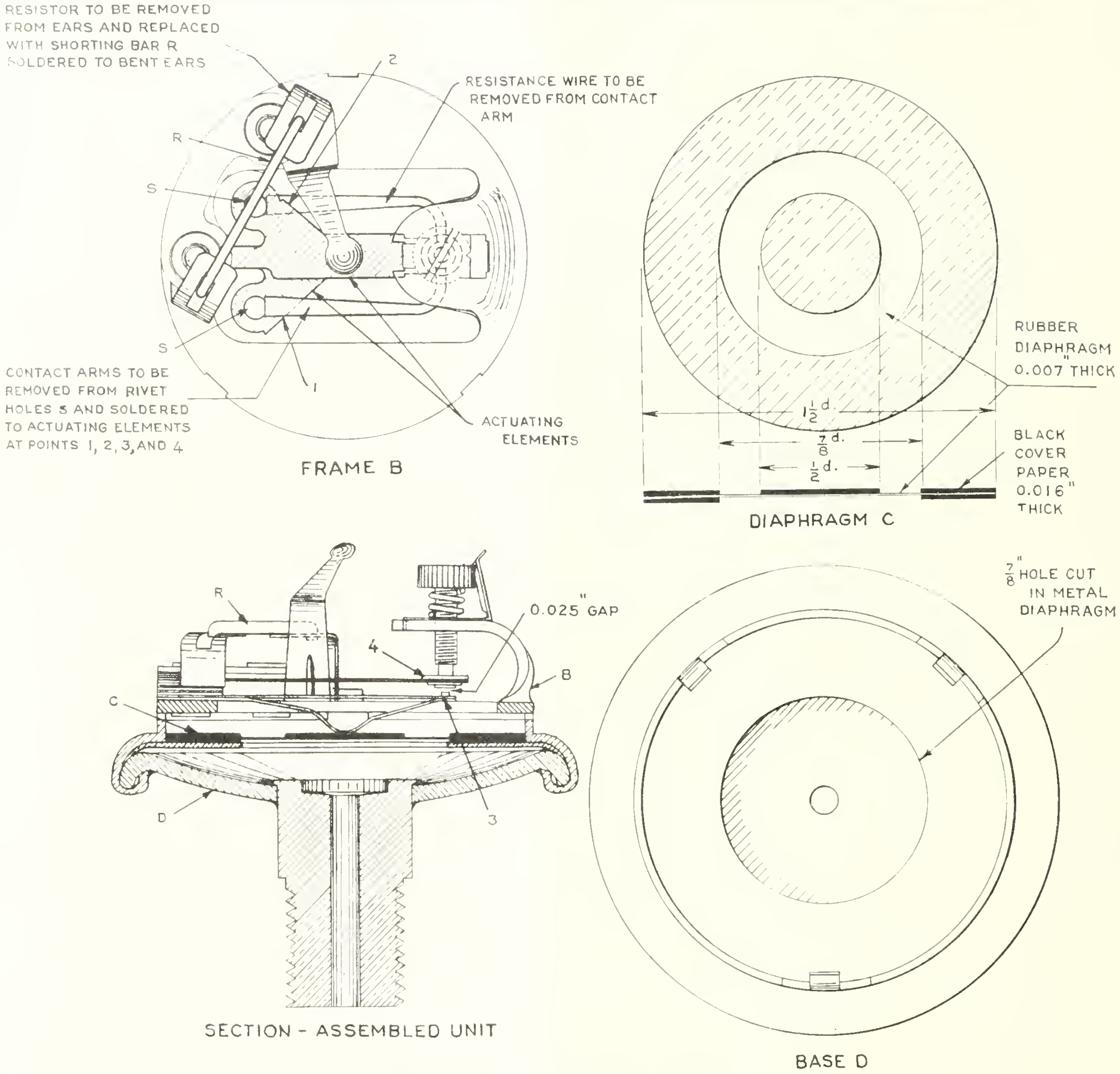
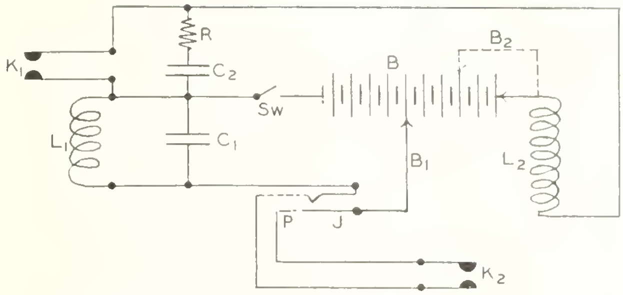

RESISTOR TO BE REMOVED FROM EARS AND REPLACED WITH SHORTING BAR R SOLDERED TO BENT EARS
2
RESISTANCE WIRE TO BE REMOVED FROM CONTACT ARM
R
S
S
CONTACT ARMS TO BE REMOVED FROM RIVET HOLES S AND SOLDERED TO ACTUATING ELEMENTS AT POINTS 1, 2, 3, AND 4
1
ACTUATING ELEMENTS
FRAME B
RUBBER DIAPHRAGM 0.007" THICK
$1\frac{1}{2}$ d.
$\frac{7}{8}$ d.
$\frac{1}{2}$ d.
BLACK COVER PAPER 0.016" THICK
DIAPHRAGM C
R
0.025" GAP
4
8
C
D
3
SECTION - ASSEMBLED UNIT
$\frac{7}{8}$" HOLE CUT IN METAL DIAPHRAGM
BASE D

Figure 4.—Details of the Road Switch.

to battery potential, and its discharge through the coil of relay L_1 upon the opening of contacts K_2 serves to increase the holding time of contacts K_1 as well as to insure operation of relay L_1 in the event of the very brief closures of contacts K_2 which result from vehicles traveling at very high speeds. Condenser C_1 also serves to suppress sparking at contacts K_2 thus helping to increase their life. Condenser C_2 in conjunction with resistor R serves as a spark suppressor for contacts K_1.

The operation of the pneumatic detector is as follows: Each pair of wheels of a vehicle in passing over the rubber tubing stretched across the highway causes an air pressure impulse that moves the rubber diaphragm in the road switch which, in turn, actuates the contacting elements, closing the battery circuit and energizing the relay in the counting unit. The end of the rubber tubing remote from the diaphragm is left open because it has been found that this permits equalization of air pressures inside and outside the rubber tubing to be quickly established after passage of the wheels over the tubing, eliminating erroneous counts resulting from air surges, as well as preventing false operation from changes in temperature.

Laboratory tests have shown that the counter will respond to impulses spaced 0.072 second apart. This speed of operation enables the counter to register both axles of a vehicle of 114-inch wheelbase traveling at speeds up to 90 miles per hour, or of a vehicle of 154-inch wheelbase traveling at speeds up to 122 miles per hour. The wheelbase of 114 inches is the average for 15 of the most popular makes of cars having wheelbases of less than 120 inches. The 154-inch wheelbase is the maximum for domestic makes of passenger cars. Al-



L$_1$ = D.C. RELAY, 200 OHM COIL

L$_2$ = D.C. ELECTROMAGNETIC COUNTER, 1,300 OHM COIL.

C$_1$ = CONDENSER, 4 MFD. 450 VOLT D.C.

C$_2$ = CONDENSER, .25 MFD. 250 VOLT D.C.

R = RESISTOR, 500 OHM, 1 WATT.

B = B BATTERIES, 90 VOLTS D C (4 - 22$\frac{1}{2}$ VOLT.)

B$_1$ = 45 VOLT LEAD B$_2$ = 67$\frac{1}{2}$ TO 90 VOLT LEAD.

Sw = SWITCH, SINGLE POLE, SINGLE THROW

J = TELEPHONE JACK.

P = TELEPHONE PLUG.

K$_1$ = RELAY CONTACTS.

K$_2$ = ROAD SWITCH CONTACTS

FIGURE 5.—WIRING DIAGRAM FOR PORTABLE AUTOMATIC TRAFFIC COUNTER.

though the counter, in most instances, will not be called upon to register vehicles traveling at such high speeds, the rapid response enables the counter to register practically all axles of passing vehicles.

The complete counter, including batteries, pneumatic detector, and connecting cable, weighs 16 pounds. The counting unit alone weighs 12 pounds. The counter was assembled from commercially available parts at a net cost of approximately $13 excluding labor.

TESTS SHOW PORTABLE COUNTER TO BE REMARKABLY ACCURATE

Once the problem of providing a properly balanced circuit for the operation of a counting unit by the road switch for a range of vehicle speeds was solved, the counter was taken into the field for testing under actual operating conditions. The first tests were made on paved roads, and the accuracy of the machine counts was entirely satisfactory. Table 1 shows the results of a typical count made on a paved road. The over-all error for the period of the count was −1.7 percent or less than the usual error for a photoelectric counter operating under the same conditions.

Table 2 gives the results of a test made to compare the accuracy of the portable counter with that of the fixed type photoelectric recorder under identical conditions. The over-all error of the portable machine was +0.3 percent, while that of the fixed type was +1.1 percent.

As a test of the sensitivity of the device, a tube 50 feet long was attached to the road switch and was installed on a 22-foot roadway so that the switch was 28 feet from the edge of the pavement. Enough tension was applied to the tube to stretch it to 52 feet. The results of this test are shown in table 3. The over-all error for the period of the count was −1.8 percent and the nearest wheel of approximately half of the vehicles counted was roughly 40 feet from the diaphragm. While an accurate check by lanes was not made, the machine appeared to count as accurately in the far lane as in the near one.

TABLE 1.—*Comparison of machine counts and manual counts of axles to check the accuracy of the portable counter*

Time (p. m.)	Axles counted—		Error
	Manually	By machine	
	Number	*Number*	*Percent*
2:07–2:12	100	100	0
2:12–2:19	137	139	+1.5
2:19–2:25	225	220	−2.2
2:25–2:38	344	338	−1.7
2:38–2:49	254	243	−4.3
2:49–2:59	281	273	−2.8
2:59–3:10	306	302	−1.3
3:10–3:24	379	374	−1.3
3:24–3:33	266	261	−1.9
3:33–3:36	126	126	0
3:36–3:40	124	122	−1.6
3:40–3:45	176	175	−0.6
2:07–3:45	2,718	2,673	−1.7

TABLE 2.—*Comparison of counts made by both the fixed and portable traffic counters with manual counts of traffic made to check their accuracy*

Time (p. m.)	Manual count			Portable counter			Photoelectric counter	
	Tractor-truck semi-trailers and other 3-axle vehicles	Total number of axles	Total number of vehicles	Number of axles counted	Number of vehicles indicated [1]	Percentage of error [2]	Number of vehicles counted	Percentage of error
2:45–3:05	0	178	89	178	89	0	89	0
3:05–3:15	1	95	47	90	45	−4.3	47	0
3:15–3:30	1	133	66	133	66	0	67	+1.5
3:30–3:45	2	166	82	166	83	+1.2	82	0
3:45–4:00	3	167	82	167	83	+1.2	85	+3.7
2:45–4:00	7	739	366	734	367	+0.3	370	+1.1

[1] Assuming each vehicle has 2 axles.
[2] On the basis of vehicles indicated.

TABLE 3.—*Comparison of machine counts and manual counts of traffic, made to check the accuracy of the portable counter when the air switch was installed 28 feet from the edge of the pavement*

Time (p. m.)	Axles counted—		Error
	Manually	By machine	
	Number	*Number*	*Percent*
2:25–2:35	303	305	+0.7
2:35–2:45	256	253	−1.2
2:45–3:00	449	440	−2.0
3:00–3:10	411	409	−0.5
3:10–3:20	339	330	−2.7
3:20–3:30	246	232	−5.7
3:30–3:40	381	373	−2.1
3:40–3:50	431	424	−1.6
3:50–3:57	431	423	−1.9
2:25–3:57	3,247	3,189	−1.8

The counter was also tested for accuracy in counting vehicles traveling at various speeds, and was found to record axles satisfactorily at speeds from 5 to 70 miles per hour. No attempt was made to test the equipment on the road beyond this range of speed, although laboratory tests indicate that the average car will be recorded properly at speeds as high as 90 miles per hour.

The first manual check made on the pneumatic detector and counter installed on a gravel road in the same manner as previously used on paved roads showed it to be in error by −21.2 percent. General observations at first indicated that the inaccuracy resulted from the wheels of vehicles passing over the tube at points where it was raised above the roadway because of an uneven cross section. However, when a test vehicle was driven so that the wheels passed over the tube at points where it rested on the surface and then over points where the tube was raised above the surface, the results showed that the entire error was not attributable to this single cause. Although the error was less when the test vehicle passed over the tube at points where it rested on the road, the error was still much greater than had been obtained on paved roads.

Considerable experimenting with different methods of installing the tube on roads having different cross sections and profiles led to the discovery that, where the tube was installed very loosely so that it would conform almost exactly to the cross section of the road, the accuracy of the counts obtained was satisfactory. It was also noted that the longitudinal profile of the road appeared to have more to do with the accuracy of the count than had the cross section. Counts made on roads having uneven longitudinal profiles showed larger percentages of error than counts made on roads with smooth profiles and very irregular cross sections. For example, counts made on a gravel road having a fairly well-shaped cross section but with a slight washboard shape along the longitudinal axis showed an error of −22.7 percent, whereas counts made on a gravel road with a grass-grown strip between the wheel tracks which kept the tube 3 or 4 inches above the tracks showed an error of −2.8 percent. These results were obtained with considerable tension in the tube. With the tube installed as recommended, the error was −7.2 and 0 percent, respectively. The practice since these tests has been to install the tube loosely, taking care not to install it so loosely that traffic will whip it out of line sufficiently to produce extra counts by the wheels on one axle passing over the tube at different times.

The number of vehicles that may be counted with one set of batteries is not definitely known, but it appears that they can be depended upon to count at least 100,000 vehicles when used where the traffic is of such volume that the batteries lose their charge through use rather than deterioration. In one test, the batteries counted approximately 120,000 vehicles over a period of 5 weeks before it was necessary to replace one of them.

PORTABLE COUNTER EASILY AND QUICKLY INSTALLED

Tests of the wearing qualities of tubing installed on paved roads indicate that the different commercial tubings that will operate the counter have a wide range of life. The poorest tubing withstood the passage of approximately 18,000 vehicles, while from accelerated tests it was calculated that the best grade of tubing tested would count three or four hundred thousand vehicles before wearing out. Thus far, in actual road tests the tubes generally have been cut by traffic rather than worn out. Under average conditions, a good grade of windshield wiper tubing has been found to count between one and two hundred thousand vehicles before failing. These failures did not result from frictional wear or the effects of repeated impacts, but rather from the tube being cut or pierced by small stones held in the tread of tires, or from some other unusual circumstance.

It is desirable to obtain as dependable a tube as possible and replace it whenever there is any reason to believe it may fail in the near future, because failure of the counter to operate for the full period will render the count up to the time of failure valueless, thus disrupting the counting schedule as well as requiring at least one extra trip to the location to obtain the desired information. This is not the case, of course, should such a detector be used with a counter that is supplemented with apparatus for periodically recording the counts, since the record would reveal the period of satisfactory operation and the data for this period would be useful.

The counter has operated satisfactorily under all of the weather conditions to which it has been exposed. It is, however, necessary to protect the open end of the tube from rain, because the suction resulting from the displacement of air by passing vehicles may draw water through the entire tube into the road switch and cause faulty counting. Water thus sucked through the tube has also caused the cement holding the diaphragm to fail. This difficulty can be overcome by placing the open end of the tube in a can similar to that used for housing the road switch. It is necessary to leave a small hole in the can so that the effect of having the end of the tube open will not be lost. Other means could also be readily devised.

The installation of the pneumatic detector and counter is very simple and can be accomplished by one man in less than 10 minutes. For best results, the most uniform cross section and profile in the section of highway for which the count is desired should be selected, and the counter placed at a point suitable for locking it to a telephone pole, tree, or other fixed object to prevent theft of the equipment. It is also advisable to select as inconspicuous a place as possible to avoid attracting attention and to reduce the possibility of tampering.

The tube is placed across the road, anchored at each end, and proper connections are made to the cable connecting the road switch and counter. As no tests have as yet been made on the equipment installed on appreciable grades, it is considered advisable to install the equipment on fairly level sections of road.

While this counter and pneumatic detector have proved satisfactory under the limited tests to which they have been subjected, the counter as now developed is not suitable for all types of traffic counting work. In fact, the device does not provide for the printing of subtotals in any way, and therefore its use must be limited to those locations where only the total volume for a given period is desired. Moreover, the detecting unit, as previously mentioned, was adapted from the means successfully used in recording vehicle passing movements. In these passing studies, the detectors were in place only while the studies were in progress and, therefore, were constantly attended and their operation could be continually checked. Any failures in their operation were immediately discovered and the detectors repaired or replaced as necessary. Although failures were infrequent, and not once during the passing studies did a vehicle cut one of the tubes, the thin tubes are believed to be particularly vulnerable to steel-tired vehicles, which will be found in greater proportion on secondary roads than on main highways.

USE OF OTHER TYPES OF DETECTOR BEING CONSIDERED

Consideration has been given to the substitution of a different type of detector even though such a change would require modifications in the electrical constants of the counting circuit. Another type of detector, employed by the Bureau in studies of the lateral placement of vehicles on the pavement, has been considered as a substitute for the pneumatic detector, although its construction is more expensive. This detector is a so-called positive-contact type in which steel strips, placed at the top and bottom of a rubber arch, are held apart by the resiliency of the rubber. The rubber arch is encased in a rubber housing for protection from the weather. As the wheels of the passing vehicles depress the arch and bring the steel strips into contact with one another, an electrical circuit is closed and, in turn, a counter is actuated.

Although this detector might not be any more durable than a rubber tube, its construction makes it less susceptible to failure resulting from cutting or abrasion. Even though a vehicle should cut through the rubber housing, it is unlikely that the steel strips would be harmed. Although a cut or fracture might permit the entrance of water which eventually would short-circuit the detector, it is probable that the trouble would be discovered in the course of regular inspections before the apparatus ceased to function. The rubber tube, on the other hand, would fail to function immediately on being cut or punctured.

Periodic recording type counter.—While the development work completed thus far has included only the simple accumulating type of counter, other agencies have been working on the design and construction of portable counters and have concentrated on the development of devices of the periodic recording type. One of these counters has been tested by the Bureau of Public Roads and at one location was found to have an error of −0.8 percent in counting 858 vehicles. The detector used for this machine consists of two strips of spring steel enclosed in a rubber casing. These strips are forced together by the wheels of passing vehicles, thus closing the electrical circuit operating the counter. The machine is timed by an 8-day spring clock which advances the record tape and closes electrical contacts, causing the machine to print the cumulative counter reading on the tape once each hour.

The Texas State Highway Department has developed a portable periodic recording counter in which the detecting is accomplished by photoelectric means. Since power is derived from batteries, the counter is readily portable. To provide for recording the counts, a cheap miniature camera using 35 mm film has been employed. An 8-day clock, suitably geared to the winding spool, moves the film ahead slowly, and by closing an electric circuit each hour, lights a small bulb thus photographing the counter at regular intervals for periods up to a week.[1]

The Bureau has also been experimenting with photographic means of recording the counts as obtained by the simple accumulating counter just described. The apparatus used also appears to offer good possibilities of recording the counter readings cheaply and accurately. It is believed that periodic recording counters, particularly when used in conjunction with a number of counters of the simple accumulating type, would be quite valuable in the continuing planning surveys for determining the general trends of traffic flow, and for further study of traffic patterns, particularly on the more lightly traveled roads.

At this early stage in the development and use of automatic traffic counters, it is felt that present knowledge is insufficient to justify a definite statement of the requirements in the field of portable automatic traffic counters. It is, however, generally agreed that there is a need for both the periodic recording and simple accumulating types of machines. Past experience indicates that the following characteristics are essential to a satisfactory periodic recording type counter provided, of course, that they all are obtainable without prohibitive initial cost or equipment too delicate for dependable field operation:

1. Continuous operation for 8 days without attention.
2. Reliable timing with error no greater than 5 minutes in 24 hours.
3. Accurate counting on both paved and unpaved roads.
4. Recording of the cumulative traffic total once each hour on the hour.
5. Installation to be accomplished by one man in about 15 minutes.

PRACTICABILITY OF USING SIMPLE ACCUMULATING MACHINES DISCUSSED

With periodic recording machines meeting these requirements operating at key locations a number of simple accumulating counters could be placed at selected locations in the area under study, noting the time and counter reading when each machine was placed in operation and the time and reading after the desired counting period. These short counts could be expanded by comparison with the continuous hourly records obtained by the periodic recording machines. This method of expanding short counts, in addition to being much easier than methods previously used, would give more reliable results. The machines might be moved every day, and they could be used to obtain week-end counts at more important stations. Another plan would be to assign one man to operate three groups of machines so that he could move each group after a week's record had been obtained. Under this schedule he would remove machines in a single group one day and reinstall them at new locations the next day.

For purposes of determining the hourly fluctuations in traffic volume, the simple accumulating type of machine is, of course, not practicable. However, the daily traffic patterns can be determined with these machines if the accumulated totals on the counters are read at 24-hour intervals. Under certain circumstances such a procedure would be entirely practicable, but if it becomes necessary in conducting any series of counts to locate the machines at some distance from one another, it may be difficult to arrange an observer's schedule so that he can read each counter within a few minutes of the specified time each day. All these difficulties are, of course, eliminated with the use of the periodic recording counter since hourly readings of the counter will be recorded from the time the counting unit is installed. But if only the total volume figures for relatively long periods, such as a full week, are required, the simple accumulating counter would be quite satisfactory.

The most appropriate type of machine to use for a particular purpose depends on the type of information required and on the relative cost of the different types of machines under the particular circumstances involved. Although the periodic recording type of machine may have a considerably higher initial cost and perhaps even a higher operating cost, these items may be more than offset by the expense involved in reading the simple accumulating counters at the required intervals. Only by experience in the field with the two types and thorough tests to determine the length of life over which each may be expected to operate effectively can reasonable unit cost be determined. Preliminary estimates, using assumed unit costs, indicate that to meet the most probable requirements of the continuing phases of the highway planning surveys, the periodic recording type machine may be cheaper to operate even at a considerable differential in original cost over that of the simple accumulating counter. The factor of mileage traveled in the use of the two types will probably be the most significant factor in the total cost of operation and, therefore, it is likely that the relative economies will differ in the various States with the wide differences in average distance between station locations which local conditions will require.

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[1] The November 1938 issue of Roads and Streets carries a description of this machine. The Texas State-wide Highway Planning Survey has also published a pamphlet describing the machine in detail.

MODIFICATIONS OF SIMPLE ACCUMULATING TYPE COUNTER SUGGESTED

Modified simple accumulating type counter.—Thus far, consideration has been given to but two types of counters and to their relative merits for use in the continuation of the highway planning surveys. However, the estimated high initial cost of an hourly recording counter has led to the consideration of special-purpose machines of simpler design. Preliminary work already begun on these counters indicates that they may be, under many circumstances, more efficient than either of the other types. An intermediate type between the periodic recording and the simple accumulating types is a counter that may be started and stopped at predetermined times. A device to obtain this type of count can readily be attached to the simple accumulating counter by inserting, in the detector circuit, a clock mechanism wired to make and break the circuit.

The Bureau is now investigating available devices, to determine whether any will satisfactorily fulfill this purpose, and is considering experimenting in the development of such a device in its own shop. With an attachment of this sort, it would be possible to install a counter, for example, in the afternoon, have the count begin at midnight, and continue for a given number of hours or days depending upon the design of the mechanism and the requirements of the count. Such an attachment should be reasonable in price and so designed that it can be employed, if desired, with any simple accumulating counter. By designing the counters and the time-control mechanisms as independent units, the use of several counters with a smaller number of timing devices would permit the adoption of extremely flexible counting schedules, and reduce to a minimum the cost of travel and labor in collecting large amounts of usable data.

The simple accumulating type of portable counter could be modified by the use in a single counter of not one but several of the electromagnetic counting devices. By attaching the time-control mechanism, modified somewhat in its operation, to this counting unit the volumes for successive periods would be registered on successive counters. Instead of stopping the count after 24 hours, the time-control device would simply shift the circuit from one counting device to the next, and so on, until the cycle was completed. Use of eight counting devices in one counter would be advantageous in that an 8-day spring-clock mechanism could be employed in conjunction with them to provide an effective complete unit.

At least two variations in the use of this 8-bank counter and time-control combination are possible. First, the time interval may be varied by certain modifications of the timing mechanism circuits to permit the accumulation of volumes during periods other than of 24 hours. Twelve-hour or 6-hour periods can be readily obtained in this way, and by a somewhat greater modification of the device, 1-hour periods might also be separately registered. Eight counters would thus permit records by 12-hour periods for 4 days, by 6-hour periods for 2 days, or 1-hour periods for 8 hours (the present generally used counting period for manual counts).

The second modification of this combination would permit the accumulating, on individual counting devices, of the total volume during corresponding periods on successive days. In this manner, the volume count would be accumulated for the desired periods for any number of days up to 8, assuming an 8-day spring clock is used. Such volume figures have not been generally utilized, but their value for control purposes is obvious.

With all the counters thus far described, it has been possible to count or to record only the total traffic using the roadway. Furthermore, these machines, as well as the photoelectric type now in general use, are poorly adapted to counting traffic on roads greater than two lanes in width. Although, with the pneumatic detector, vehicles in two lanes moving in the same direction have been counted with reasonable accuracy in light traffic, there is reason to believe that considerable error may be introduced by vehicles crossing the tube simultaneously under heavy traffic conditions on multilane roads.

This source of error may be eliminated by utilizing the positive-contact detector previously mentioned. For this purpose, however, but one of the steel strips would be continuous over the entire length of the detector. The other would be broken into sections and leads from each section taken to an individual counter, using one section and, correspondingly, one counter for each lane of traffic. A combination of this type, therefore, suggests a further use for a counting unit containing a bank of counters.

Although lane counts have not generally been made in conjunction with the highway planning surveys, it is considered desirable, particularly in the determination of proper widths of multilane highways, to have an accurate knowledge of the direction of traffic flow as well as of its total volume. This detecting and counting unit would also be ideally adapted to city traffic survey work where directional flow and the volume per lane are essential in the design of traffic control methods and in general traffic regulation.

The need for portable automatic traffic counters has long been felt, not only by State and Federal highway officials, but also by city traffic officials and by various commercial agencies to whom a knowledge of traffic volume is essential. In practically all instances the demand has been evidenced by attempts to design equipment capable of performing the types of traffic counts now generally conducted by manual methods. However, a factor which should always be kept in mind in considering the design of traffic counters is that it is entirely possible that many types of counts, previously impossible or impractical by manual methods, may be quite readily accomplished by properly designed automatic counters. It is, therefore, essential in the design of counting mechanisms that the instrument designer work in close coordination with the traffic engineer in order that equipment developed will possess the greatest possible flexibility. Similarly, the traffic engineer is remiss if, in his design of counting schedules, he fails to avail himself of the possible advantages of flexibly designed automatic devices.

In describing the development work to date, it is desired only to present a progress report and to make suggestions that may assist in continuing development work to provide devices most suitable to their individual requirements.

(Continued from page 212)

It is only by analysis of the normal driving practices, as they are recorded for a wide variety of driving conditions, that it will be possible to determine whether the problem is one presented by the average driver, or whether present highway design practices are creating entirely unnecessary problems, either impossible or extremely difficult of solution. It is this phase of the problem to which the greatest attention must be directed. Determination of the distances involved in passing maneuvers is a simple, even though laborious, matter. The determination of the effect of highway alinement and of driver psychology upon future design requirements is a matter requiring far more comprehensive research.

Although this report has been devoted to the methods employed in the analysis of passing practices, it will be evident that the data collected in these field studies will permit of a variety of analyses quite distinct from this one, rather narrow, use. The advantage of this procedure, particularly with respect to the field work, is that the actual occurrences on the entire section under observation have been recorded completely and continuously throughout the study periods in a permanent and easily interpreted form. Further analysis can be conducted simply by referring to the original field charts, or to the transcribed records.

BIBLIOGRAPHY ON HIGHWAY FINANCE NOW AVAILABLE

A bibliography on highway finance has recently been prepared by the Bureau of Public Roads of the United States Department of Agriculture and mimeographed copies are now available for free distribution.

The bibliography is selective in character and includes references to books, articles printed in technical and other periodicals, and publications of societies. It covers chiefly the period from 1928 to date, but includes references to earlier material published by the Bureau of Public Roads.

Librarians, students, and research workers will find the bibliography a valuable aid in locating published material on highway finance.

Single copies of the bibliography can be obtained, without charge, from the Bureau of Public Roads, United States Department of Agriculture, Washington, D. C.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF DECEMBER 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 3,558,906	$ 1,685,635	112.4	$ 8,857,711	$ 4,417,375	367.8	$ 1,323,380	$ 660,385	54.2	$ 1,988,046
Arizona	1,931,132	1,474,674	107.4	876,305	563,704	24.9	437,725	194,213	30.6	1,072,275
Arkansas	1,036,098	1,028,697	73.7	3,056,895	3,053,158	191.1	915,470	914,409	53.1	231,186
California	7,660,168	4,213,500	181.4	6,634,208	3,507,830	94.2	1,850,283	983,669	50.2	1,145,410
Colorado	2,272,872	1,229,934	84.3	3,030,903	1,592,037	100.0	751,260	417,710	13.0	1,491,940
Connecticut	934,030	455,835	8.9	684,518	337,455	7.9				1,525,005
Delaware	481,532	237,550	14.1	712,036	353,260	9.9	259,491	125,600	6.6	1,137,707
Florida	1,729,200	864,600	39.8	3,106,238	1,553,119	57.1	277,000	138,500	3.9	2,812,495
Georgia	4,297,272	2,104,079	216.2	4,905,350	2,452,675	232.7	1,036,020	518,010	71.2	5,262,724
Idaho	2,068,519	1,222,347	200.7	1,189,537	710,407	39.0	301,398	179,452	10.9	925,222
Illinois	10,102,553	5,047,048	263.1	7,594,236	3,793,494	166.1	3,530,611	1,765,260	84.7	977,887
Indiana	4,446,630	2,181,093	127.3	3,836,570	1,918,285	70.8	1,574,584	787,212	37.7	2,168,782
Iowa	7,171,962	3,344,519	234.0	5,121,839	2,165,983	161.9	540,887	126,000	34.8	56,547
Kansas	4,302,078	2,149,829	673.6	4,362,243	2,181,121	211.1	3,386,922	1,686,436	150.2	2,273,945
Kentucky	5,584,169	2,775,513	199.3	2,680,426	1,340,213	57.7	1,306,746	653,373	39.1	1,996,080
Louisiana	1,294,187	646,891	38.2	11,742,753	2,516,164	30.3	1,761,038	775,592	28.6	1,782,072
Maine	2,715,195	1,353,191	62.6	1,590,201	795,100	35.2	227,424	113,711	1.9	146,583
Maryland	1,085,456	542,728	17.1	2,094,767	1,031,621	40.6	1,148,731	564,730	10.4	1,540,916
Massachusetts	1,863,674	931,834	9.0	2,984,200	1,491,641	19.4	649,117	322,815	8.4	2,198,362
Michigan	7,424,383	3,599,839	156.2	3,982,708	1,990,702	117.9	1,638,625	817,580	39.5	808,471
Minnesota	4,780,350	2,314,905	289.3	6,086,875	3,023,245	279.3	1,153,760	575,975	61.2	1,904,038
Mississippi	1,802,608	874,442	71.2	10,282,812	3,919,027	447.0	1,610,990	622,220	68.3	1,827,603
Missouri	4,213,623	1,971,236	118.8	2,565,348	1,272,436	58.1	5,754,788	2,742,950	222.8	2,222,644
Montana	1,666,619	936,796	83.6	756,924	425,407	17.6	1,012,152	569,320	69.9	3,882,156
Nebraska	3,204,113	1,565,053	309.3	5,278,728	2,662,277	400.2	5,226,354	1,981,124	390.2	894,856
Nevada	1,447,314	1,241,763	168.8	1,085,933	935,776	38.5	257,545	223,292	12.6	697,909
New Hampshire	881,761	437,187	21.5	437,424	217,752	3.3	93,802	46,900	1.4	1,145,923
New Jersey	1,132,455	557,970	10.8	2,824,716	1,409,248	18.9	1,097,710	548,255	9.8	1,965,117
New Mexico	2,054,234	1,252,372	237.1	2,132,020	1,400,484	74.8	204,244	91,230	33.4	344,832
New York	12,919,513	6,358,010	233.9	10,361,696	5,135,342	170.0	3,822,030	1,600,760	63.6	194,598
North Carolina	6,082,253	2,917,224	248.9	5,036,079	2,516,882	323.1	875,580	425,400	44.8	1,585,065
North Dakota	3,319,408	3,186,988	254.8	500,901	290,805	57.5	69,522	37,236	6.8	3,557,402
Ohio	7,108,523	3,513,044	86.4	8,056,842	4,018,822	78.2	1,978,040	985,864	18.7	5,415,111
Oklahoma	4,852,873	2,562,706	206.1	3,489,873	1,803,756	97.5	1,894,300	999,863	52.0	2,167,924
Oregon	3,131,747	1,849,730	110.7	1,337,972	816,987	83.2	610,573	372,460	26.6	1,382,171
Pennsylvania	7,869,759	3,908,555	132.6	6,937,785	3,466,314	72.1	3,584,605	1,737,299	32.7	1,829,173
Rhode Island	1,179,290	589,645	16.4	372,212	186,106	3.5	307,320	153,660	3.2	907,105
South Carolina	3,807,183	1,671,548	230.6	3,808,045	1,712,476	92.6	580,751	264,300	29.5	1,160,855
South Dakota	1,883,737	1,040,551	228.5	4,675,086	2,585,440	446.3	495,660	274,080	45.4	2,515,736
Tennessee	4,216,795	2,107,676	147.0	3,108,609	1,555,026	56.4	1,564,340	781,670	30.6	3,546,575
Texas	10,739,595	5,309,928	720.6	11,224,350	5,537,497	472.2	4,965,291	2,413,406	293.9	3,530,777
Utah	1,023,819	723,724	112.0	2,000,988	1,418,640	68.2	207,275	147,580	9.9	528,029
Vermont	1,283,285	582,204	34.9	722,784	343,793	17.7	196,970	98,295	4.3	169,425
Virginia	5,915,394	2,955,516	204.9	2,861,368	1,428,849	88.0	903,848	449,619	10.9	362,304
Washington	3,989,159	2,083,068	99.5	2,600,727	1,364,359	25.9	416,525	216,200	10.2	570,311
West Virginia	1,646,147	1,183,551	61.6	1,333,822	703,786	26.9	911,370	451,835	27.2	1,947,582
Wisconsin	3,999,467	1,955,828	134.0	6,744,767	3,186,680	188.6	951,427	435,400	7.7	1,002,426
Wyoming	2,420,989	1,488,150	279.2	313,782	188,371	38.1	339,430	209,730	38.2	496,062
District of Columbia										
Hawaii	824,027	408,939	17.7	826,430	405,695	9.5	28,750	11,410		1,205,177
Puerto Rico	53,980	26,990	2.1	1,139,989	567,815	22.6	598,946	297,050	12.1	342,520
TOTALS	181,410,036	94,664,635	7,692.1	187,949,031	92,264,937	5,811.4	64,630,660	31,509,540	2,366.9	80,843,061

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF DECEMBER 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 197,900	$ 98,950	11.9	$ 533,605	$ 265,150	37.3	$ 314,700	$ 156,800	7.9	$ 532,072
Arizona	281,361	187,459	18.6	176,900	115,539	17.5	7,440	5,357		348,127
Arkansas				124,036	116,677	12.1	269,518	267,911	28.4	479,520
California	1,032,491	587,924	78.6	950,335	522,097	50.1	499,627	265,530	19.0	472,285
Colorado	650,358	358,164	41.0	555,510	288,585	29.4	38,040	15,000		259,819
Connecticut	48,450	24,205	1.0	62,584	31,132	.5	73,100	17,455	1.1	245,486
Delaware	18,950	9,475	4.1	53,394	26,697	8.8	47,240	23,135	9.9	187,568
Florida				254,204	127,061	1.5	153,500	76,750	4.4	471,041
Georgia	225,939	109,021	29.4	610,586	305,293	76.2	186,360	93,180	26.4	751,228
Idaho	451,521	204,347	46.9	64,986	27,513	10.3	128,534	76,572	2.6	165,939
Illinois	1,257,586	628,025	112.7	1,661,092	776,546	93.5	377,700	180,350	30.5	443,904
Indiana	599,857	255,400	64.8	679,900	322,150	62.0	703,248	341,752	62.8	323,136
Iowa										1,298,449
Kansas	53,582	26,791	8.6	206,472	103,235	13.3	100,686	50,343	34.6	1,138,602
Kentucky	706,675	209,989	93.3	785,052	216,671	35.8	779,443	229,948	88.6	162,187
Louisiana	64,068	31,900	6.9	505,518	224,125	38.6	567,914	252,510	48.2	215,601
Maine	362,500	181,250	23.3	262,662	126,214	12.5	18,700	9,350	1.2	11,759
Maryland				45,164	15,787	5.8	292,800	110,355	18.4	286,334
Massachusetts				134,123	66,945	2.4	222,970	110,905	4.9	468,550
Michigan	349,961	173,481	32.0	638,804	319,402	41.1	483,200	240,850	23.8	801,806
Minnesota	283,896	131,799	42.2	452,110	224,011	38.6	374,824	187,412	23.9	836,358
Mississippi				299,000	149,500	23.8				739,427
Missouri	320,119	153,359	43.1	147,040	73,130	11.5	895,460	386,430	115.7	452,753
Montana				13,983	7,865					1,027,170
Nebraska	435,709	215,851	79.8	391,214	193,127	59.7	558,656	272,107	100.8	316,731
Nevada	424,798	354,271	68.8	109,464	91,876	15.0	34,544	29,956	2.0	19,752
New Hampshire	241,175	119,755	6.0	64,509	31,583	2.3				95,537
New Jersey	123,040	61,520	2.4	199,860	91,195	2.7				520,058
New Mexico	563,070	343,413	36.9	538,095	328,178	30.9	104,104	63,490	10.1	71,691
New York	2,245,354	1,117,525	159.8	1,782,800	891,400	94.7	178,400	89,200	11.4	264,369
North Carolina	606,848	303,424	72.3	867,284	433,620	66.6	117,630	54,580	13.7	290,231
North Dakota	52,020	27,860	9.0	101,510	54,367	25.8	115,886	62,065	8.5	642,960
Ohio	156,560	78,280	3.8	52,490	32,470	.9	497,100	248,550	24.3	1,472,341
Oklahoma	31,300	16,652	2.1	399,018	211,194	36.2	617,940	305,608	37.0	651,461
Oregon	453,626	266,110	58.5	59,454	36,322	5.0				452,701
Pennsylvania	1,611,605	782,562	115.6	1,795,779	880,108	97.6	549,232	274,616	31.1	179,398
Rhode Islaod	66,840	33,420	3.5	162,675	81,314	4.8	74,070	37,035	.9	36,122
South Carolina	359,615	161,872	38.3	789,615	325,779	86.4	303,400	118,500	23.6	72,529
South Dakota				11,300	6,250					816,436
Tennessee	120,720	60,360	2.9	656,244	255,222	36.9	174,320	80,690	5.5	663,218
Texas	2,109,627	987,459	306.8	1,847,614	794,250	183.7	726,281	344,323	109.0	998,738
Utah	402,522	222,870	40.1	222,529	118,625	15.2	208,420	91,460	20.5	117,415
Vermont	240,777	108,150	13.8	90,306	45,153	4.0	43,300	20,500	.5	35,793
Virginia	436,847	216,208	57.0	787,620	343,870	47.2	126,162	63,081	18.8	269,226
Washington	561,569	293,796	63.8	388,264	204,178	20.1	349,621	184,100	20.1	48,482
West Virginia	124,300	62,150	9.5	236,846	118,423	18.1				369,101
Wisconsin	373,781	173,542	12.7	815,142	399,890	42.0	23,126	11,450	.8	627,765
Wyoming	416,146	254,582	59.0	285,102	176,169	11.9	121,478	75,041	10.4	79,401
District of Columbia										
Hawaii				56,250	28,125	2.4				218,750
Puerto Rico	74,740	37,370	3.9	243,162	120,595	13.9	31,422	15,425	2.3	73,485
TOTALS	19,137,803	9,670,541	1,884.7	22,171,206	10,744,608	1,546.6	11,490,096	5,539,672	1,003.6	21,522,812

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Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

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Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

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Highway Bond Calculations. 10 cents.

DEPARTMENT BULLETINS

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SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . . Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF DECEMBER 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 243,609	$ 243,410	6		4	$ 947,379	$ 945,524	9	1		$ 57,400	$ 57,400	2		3	$ 739,185
Arizona						9,452	9,452									620,761
Arkansas	82,511	81,574	4			461,497	461,106	8			278,920	278,811	5			937,278
California	669,417	669,417	2	2		1,147,259	1,146,684	7	2		667,526	667,046	2			1,217,062
Colorado	30,569	27,337	1		1	131,999	131,999	2		1	517,478	509,348	4		17	619,628
Connecticut						18,930	12,665									831,825
Delaware	24,500	24,500			7						61,920	61,920	2		16	407,330
Florida						435,116	435,116	3								970,681
Georgia						202,880	202,880	4			183,050	183,050	2		1	2,031,457
Idaho	174,973	174,800	4			263,415	249,386	4			131,898	120,500	1			278,184
Illinois	286,500	286,500	2		1	1,950,825	1,950,825	12	2	9	870,690	870,690	6		66	2,096,925
Indiana	688,715	599,689	3	5		754,680	727,780	2	1	89	271,000	271,000	1	1		815,680
Iowa	1,027,480	972,694	10	1	19	203,820	191,700	2	1	1	140,097	130,308	7		1	1,272,859
Kansas	455,031	455,022	8		4	671,850	671,850	10			478,465	478,465	4		6	969,171
Kentucky	145,000	145,000	1			344,297	344,297	4	1	31	441,642	441,642	6	1		883,106
Louisiana						233,219	226,478	3			476,771	473,090	10			877,503
Maine	48,590	48,590	2			332,396	332,396	3	2		69,830	69,830	1		1	240,387
Maryland						64,586	64,586	1								962,247
Massachusetts	54,710	54,710		1		220,486	220,290	1	1		223,815	223,705		3		1,414,583
Michigan	893,783	887,372	8	1	16	690,796	690,796	6	2	29	114,000	114,000				1,593,017
Minnesota	39,556	39,556		1	1	760,185	759,864	3	5		18,297	18,297			3	1,628,254
Mississippi						465,800	465,800	6			250,000	250,000	1			868,351
Missouri	184,074	182,952	3	1		223,970	223,970	2			397,960	397,960	1	1		2,220,371
Montana	355,586	350,704	4			534,647	534,647	5			323,913	323,913	4			115,207
Nebraska	150,374	150,374	4			729,314	729,314	15			757,668	757,668	18		33	127,054
Nevada	149,761	149,761		3	2	203,311	203,311	2			11,280	11,280			4	94,289
New Hampshire	63,852	63,545		1	1	94,076	94,017	5		2						336,188
New Jersey	116,891	111,665	1	1		223,914	223,914	1	1	2						1,634,678
New Mexico	168,984	168,984	4	1		118,994	118,994	3								563,513
New York	992,501	991,800	4	3	2	1,720,351	1,712,101	5	7		231,660	136,520		2	1	3,929,508
North Carolina	81,950	81,950		1	1	917,500	884,800	7	5		276,760	276,760	1	1	10	1,200,884
North Dakota	184,700	184,700		1		637,438	637,438	4	1							757,083
Ohio						437,870	437,870	5			329,990	329,990	4			3,461,308
Oklahoma						17,343	17,343				359,305	325,305	2		55	1,938,861
Oregon	308,391	307,742	1	2		58,634	58,634				129,997	129,997	2			481,611
Pennsylvania	138,043	122,837	1			304,601	299,694	2			1,421,663	1,210,027	3			4,099,977
Rhode Island						335,019	335,019	1	2		103,772	103,772		1		54,959
South Carolina	20,930	20,930				164,359	109,393	3	2		335,820	335,820	12		40	1,022,959
South Dakota	119,383	119,383	2		8	286,368	286,368	3	2	1	19,820	19,820	2		2	894,529
Tennessee						44,830	44,830		1		310,680	310,680	1	1	7	1,536,523
Texas	34,033	33,377	2			1,306,764	1,305,352	12	6		566,572	508,765	7			3,495,622
Utah	101,648	101,648	2		4	47,359	47,359	2								424,056
Vermont	208,512	203,155	6	1	3	36,504	36,504		1	3	18,330	18,330			6	222,463
Virginia	248,306	248,306	12	1		290,418	290,418	4	1		348,843	348,843	5	1	13	908,463
Washington	179,118	177,018	1	3	2	747,122	747,122	9	1		111,006	109,595	1		12	351,875
West Virginia	216,320	215,300	1			200,861	200,861	1		2	147,750	131,990	4		4	775,947
Wisconsin	221,661	221,493	3			1,187,718	1,101,026	11			4,917	4,917			1	1,146,073
Wyoming	156,370	156,370		2	1	15,180	15,180			2	19,810	19,810			8	485,370
District of Columbia						34,262	34,262		1							291,169
Hawaii						193,200	193,200	3	1							300,550
Puerto Rico						214,569	213,370	4								516,930
TOTALS	9,266,332	9,074,165	102	32	77	21,637,363	21,377,785	199	50	172	11,480,315	11,030,864	121	12	310	55,663,494

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 19, NO. 12 FEBRUARY 1939

FIELD PARTY OBTAINING HIGHWAY CAPACITY DATA ON AN ILLINOIS HIGHWAY

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PUBLIC ROADS ▸▸▸ *A Journal of Highway Research*

Issued by the

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

D. M. BEACH, *Editor*

Volume 19, No. 12 February 1939

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

PRELIMINARY RESULTS OF HIGHWAY CAPACITY STUDIES

Reported by O. K. NORMANN, Associate Highway Economist, Bureau of Public Roads

DURING the last few years many miles of highway have been reconstructed or replaced, not because they failed structurally, but because they became obsolete through inability to permit the present volumes of traffic to move at reasonable speeds and with freedom from interference between vehicles. Statewide highway planning surveys now being conducted in 46 States will furnish highway engineers with more accurate traffic volume figures and future traffic estimates than they have had in the past, but to make the best possible use of this information the capacity of various surface widths and types of design and alinement must be known.

Many attempts have been made to determine highway capacities by both theoretical formulas and field observations. In general the theoretical derivations, obtained by assuming that all vehicles travel at the same speed, have resulted in extremely high and impractical volumes. The field observations have either been too limited or lacking in essential data, such as the speeds at which the individual vehicles were traveling, to yield accurate and conclusive results.

Realizing that a large quantity of detailed data would be required before the relative capacities of highways of various widths and alinements could be determined, the Bureau of Public Roads conducted a number of capacity studies on some of the most heavily traveled rural highways in New York and New England during the summers of 1934 and 1935, and cooperated with the Illinois Division of Highways in a similar study in Illinois during 1937.

At all study locations the time that each vehicle entered and left a section of highway ⅕ mile long was entered on a graphic time-recorder (see fig. 1) and each vehicle was classified as a passenger car, bus, or truck. Each truck was further classified as a light, medium, or heavy truck, truck and full trailer, or truck and semitrailer. It was possible to obtain from these basic data the speed of each vehicle, its time or distance spacings, from other vehicles, and the exact volume of traffic in each direction during any desired time period. The studies were conducted for approximately 8 hours each day, starting while traffic was light and continuing through the heaviest volumes. This gave a range in traffic density at each location.

Data for a total of over 300,000 vehicles with volumes as high as 3,400 per hour in one direction on a four-lane road were recorded during the studies. The majority of the sections were level tangents on rural highways. Although 300,000 vehicles may seem to be a large sample, it is not to be expected that the data will yield answers to all highway capacity problems. However, a few significant traffic characteristics have been developed by the analyses thus far completed. The purpose of this report is to present a few typical results.

Figure 2 shows the tabulating machine card used for the Illinois study and illustrates the data that are obtained for each vehicle passing through the study sections. Entries in the columns headed "time at center" enable the cards to be sorted into any time period group desired.

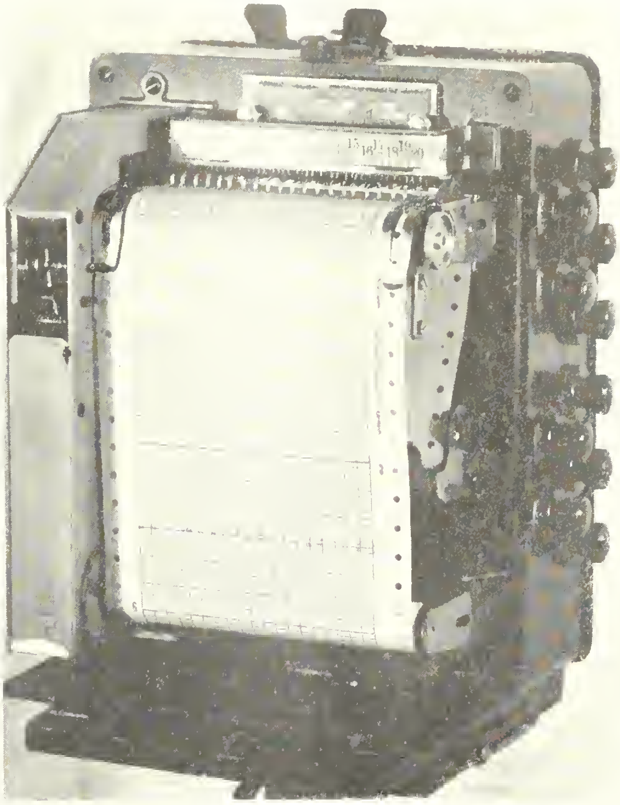

FIGURE 1.—GRAPHIC RECORDER USED IN HIGHWAY CAPACITY STUDY.

MAXIMUM TRAFFIC VOLUME WHEN ALL VEHICLES TRAVEL 33 MILES PER HOUR

The theoretical maximum capacity of a traffic lane for various vehicle speeds was first determined for one location by a method similar to that used in many of the theoretical derivations, but instead of using a calculated uniform spacing between vehicles, the actual spacings obtained by the study were used. Of the 8,500 vehicles recorded at this location, 2,055 were traveling at the same speed as the preceding vehicle and were not passed or did not pass another vehicle while in the section. By classifying these vehicles into speed groups, the modal spacing for each group was determined mathematically and checked by constructing a curve showing the frequency distribution of spacings for each speed. Figure 3 shows the frequency distribution of spacings for the 31-mile-per-hour group.

The modal spacings for all speed groups are shown in figure 4. At speeds greater than 20 miles per hour there was very little difference in the time spacing allowed by the driver in the majority of cases between his vehicle and the vehicle ahead when both were traveling at the same speed.

The distance spacings as shown in the lower curve of figure 5 are readily computed from the time spacings. The upper curve was constructed by using the average spacings for all vehicles spaced at less than 4 seconds.

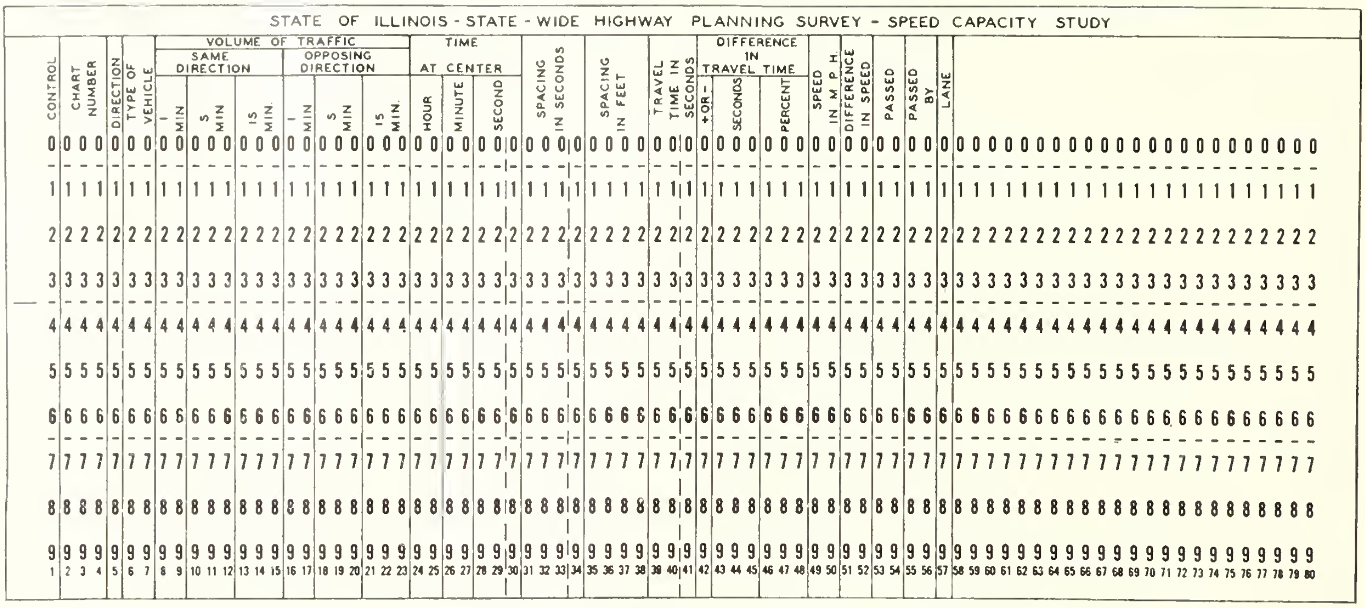

Figure 2.—Tabulating Machine Card Used in Analyzing Data Collected in Highway Capacity Studies.

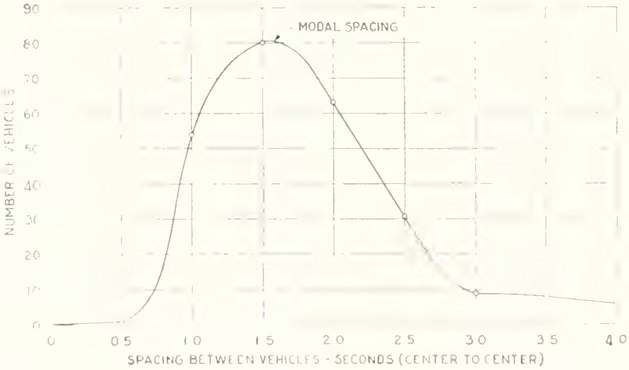

Figure 3.—Frequency Distribution of Spacings Between Vehicles Traveling 31 Miles Per Hour Preceded by Vehicles Traveling at the Same Speed. Section 3-J, 2-Lane Tangent.

It was assumed that a car traveling at a spacing greater than 4 seconds was not traveling at the minimum spacing but just happened to be going at the same speed as the preceding vehicle. These two curves, therefore, represent the distance spacings that the average driver allows between his vehicle and the vehicle just ahead when following at the same speed, and the distance allowed by the largest group of drivers under the same conditions.

It is of interest to know what percentage of these vehicles could have stopped without hitting the preceding vehicle had it suddenly stopped. If it is assumed that the brakes on the following vehicles were as good as those on the leading vehicles and that a ½-second reaction time would be required, 4.4 percent of the vehicles could not have avoided rear-end collisions. This figure increases to 28 percent for a 1-second reaction time, and to 50.5 percent when 1.5 seconds is allowed for a combined perception and reaction time.

Although the possibility of the leading vehicle stopping almost instantaneously, as in the event of a head-on collision, is rather remote, the drivers of 77.3 percent of the vehicles could not have stopped in time to avoid a rear-end collision or the necessity of swerving to the shoulder or ditch had they suddenly been confronted with such a condition. This assumes a ½-second reaction time, four-wheel brakes in perfect order, and the most nonskid type of road surface. These figures illustrate one reason why more than two vehicles are so often involved in one accident.

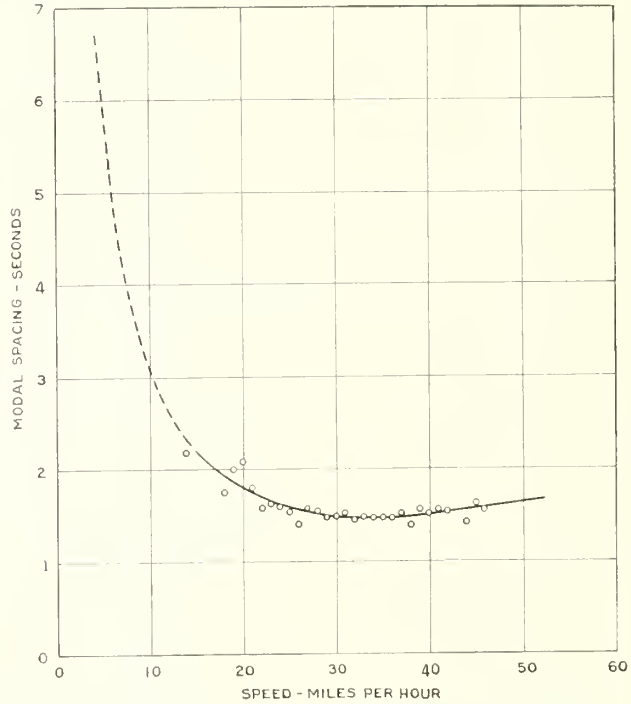

Figure 4.—Modal Spacing in Seconds Between Vehicles Traveling at the Same Speed. Section 3-J, 2-Lane Tangent.

Using the values shown in figure 5, figure 6 was constructed to show the theoretical maximum number of vehicles per hour that one traffic lane will accommodate at various speeds if all vehicles travel the same speed. The maximum volume occurs at a vehicle speed of 33 miles per hour and does not vary more than 20 percent from this volume for any speed between 20 and 50 miles per hour.

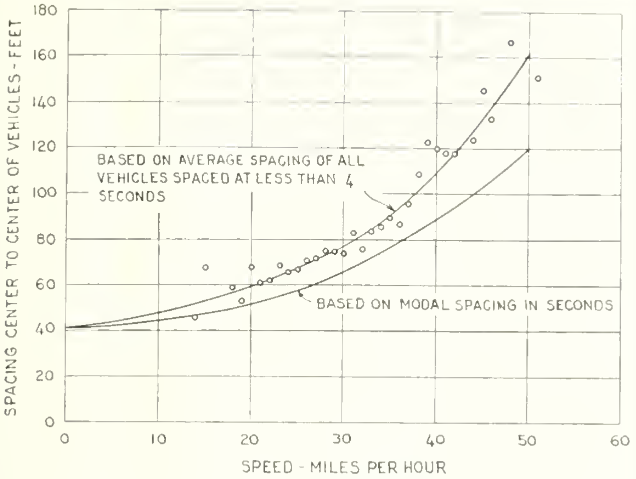

Figure 5.—Minimum Average Spacing in Feet Between Vehicles Traveling at the Same Speed. Section 3-J, 2-Lane Tangent.

Similar curves could be constructed for other sections of highway having various designs, but would be of doubtful value. They would hold true only in instances where all vehicles were required to move at the same speed—none could go faster or slower than the vehicle ahead and there could be no passing or interruptions of any kind. Such uniformity would be practically impossible to attain except in rare instances such as through tunnels or over long bridges where strict control would be possible, but certainly not on rural highways. A practical working capacity that will permit a reasonable range of speeds and a reasonable amount of freedom from interference between vehicles must, therefore, be determined. It would not even be justifiable to assume that volumes for practical working capacities at various average speeds will be proportional to the volumes shown on the curves developed by the method illustrated.

Many of the available articles on highway capacity indicate that as the volume of traffic increases, a point will be reached beyond which the average speed of the vehicles will decrease. Some indicate that there will be a sudden and appreciable drop in the speed. The majority of present definitions for maximum capacity, working capacity, practical working capacity, and the beginning of congestion, are based on the assumption that such a point does exist. To determine if there is such a point, separate curves were made using data collected at each of the 35 locations by plotting the average speed for various volumes of traffic. Data for about 180,000 vehicles were grouped in 100-vehicle groups, 15-minute groups, and 1-hour groups, and these were used together with both the total and one directional volumes. In most instances there was a gradual decrease in average speed with an increase in volume. A number of locations showed a relatively high rate of decrease in speed at the high volumes, but several showed a relatively low rate of decrease at the high volumes. The traffic volumes and densities at these locations varied from values below to values well above those that are definitely known to be in excess of reasonable working capacities for rural highways, yet none of the curves showed a sudden or marked decrease in speed at any particular traffic volume or density. The individual points for groups of vehicles in similar volumes were scattered over a relatively wide range of speeds, although the averages for groups within a limited volume range formed fairly uniform curves.

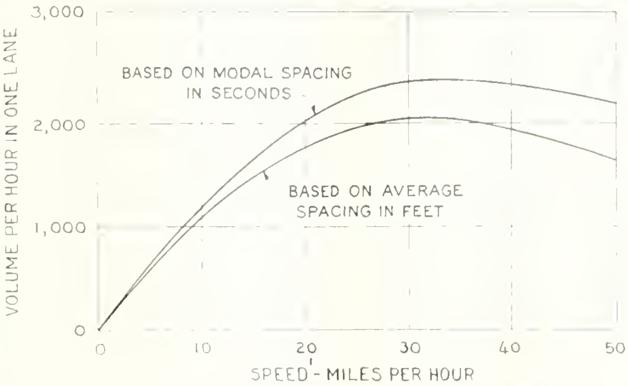

Figure 6.—Theoretical Maximum Traffic Volume With all Vehicles Traveling at the Same Speed. Section 3-J, 2-Lane Tangent.

AVERAGE SPEED DIFFERENCE USED AS AN INDEX OF TRAFFIC CONGESTION

Speed-volume or speed-density curves for traffic at various locations do not give a basis for determining either absolute or relative values for the capacities of highways or amount of interference between vehicles. It cannot be assumed that the interference between vehicles increased only 30 percent merely because the average speed dropped 30 percent with an increase in volume from 400 to 1,400 vehicles per hour at one location.

It is apparent that a measure other than average vehicle speed alone must be used to determine the working capacities of highways. Were it possible and practical to measure the physical, mental, and nervous energy used by the average driver per mile, when traveling in various volumes of traffic, this would be an ideal index for determining working capacities. As this method has not been found practical, a search for another index obtainable from available data was made.

Curves were drawn showing the standard deviation from the average speed, the standard deviation of the differences in speed from the preceding vehicle, and the actual number of passings made within a given length of highway. The most significant index found to measure the relative interference between vehicles on rural highways was the mean or average difference in speed between successive vehicles.

With light traffic on a rural highway, the speed of each individual vehicle is not governed by the speed of the vehicle immediately ahead, and there is a relatively high mean difference in speed between successive vehicles. As the volume increases, there is an increasing tendency for the speed of the individual vehicles to be governed by the speed of the preceding vehicles. This causes a marked decrease in the mean difference in speed between successive vehicles, although the decrease in average speed may be slight.

To illustrate how the index may be used as a measure of the relative interference between vehicles, figure 7 was constructed from the data for one section of a two-lane highway. The individual vehicles were classified into groups as determined by the spacing in seconds from the preceding vehicle. The average speed and the mean difference in speed between successive vehicles were then obtained for each group. This figure indicates: (1) That operators of vehicles at or above a time spacing of 9 seconds from the preceding vehicles were not influenced by the speed of the preceding vehicle; (2) at spacings below 9 seconds some of the operators

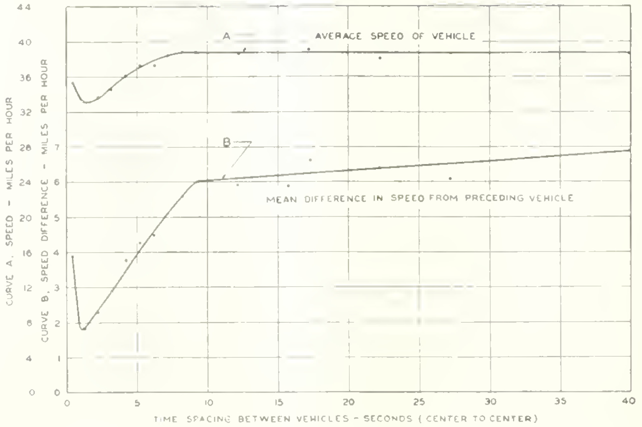

Figure 7.—Speed Characteristics of Vehicles Traveling at Given Time Spacings Behind Preceding Vehicles. Section 3-J, 2-Lane Tangent.

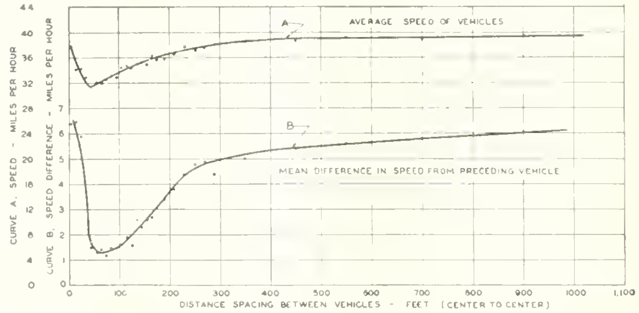

Figure 8.—Speed Characteristics of Vehicles Traveling at Given Distance Spacings Behind Preceding Vehicles. Section 3–J, 2-Lane Tangent.

were influenced by the speed of the preceding vehicles; and (3) at a spacing of 1½ seconds, a large majority of the drivers were influenced by the speed of the preceding vehicle. The increase in the average speed and difference in speed for spacings below 1½ seconds was undoubtedly caused by vehicles passing one another.

Similar curves were constructed using only the vehicles traveling as fast or faster than the preceding vehicle. Except that the speed curve was about 2 miles per hour higher for values above 9 seconds, the curves were identical with those of figure 7.

Figure 8 was constructed in the same manner as figure 7, except that distance spacing groups were used instead of time spacing groups. The same general characteristics are evident. Although the traffic speed curves are considerably higher for some locations than for others, the break in the curves occurs consistently at about 9 seconds when time spacings are used but varies widely for the distance spacing curves. This indicates that the average driver starts to be influenced by the speed of the preceding vehicle at a fairly constant time spacing or at a distance spacing that varies with his speed.

On a two-lane highway, the average vehicle spacing is 9 seconds with 400 vehicles per hour in each direction. There would be no interference between vehicles at this volume if all vehicles were uniformly spaced and traveled at the same speed. Because of the large variation in individual vehicle speeds, the time spacings between vehicles are continually changing so that even with traffic volumes considerably lower than 400 vehicles per hour, a large percentage of the vehicles travel at spacings much shorter than 9 seconds. The percentage of vehicles traveling at the shorter time spacings will obviously increase with an increase in volume.

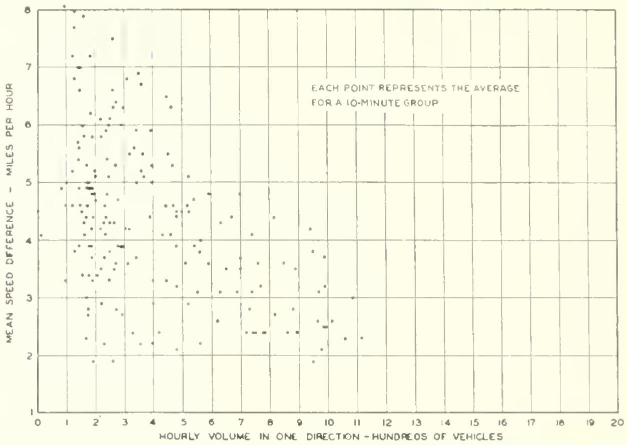

Figure 9.—Mean Difference in Speed Between Successive Vehicles for Various Volumes of Traffic. Section 3–J, 2-Lane Tangent.

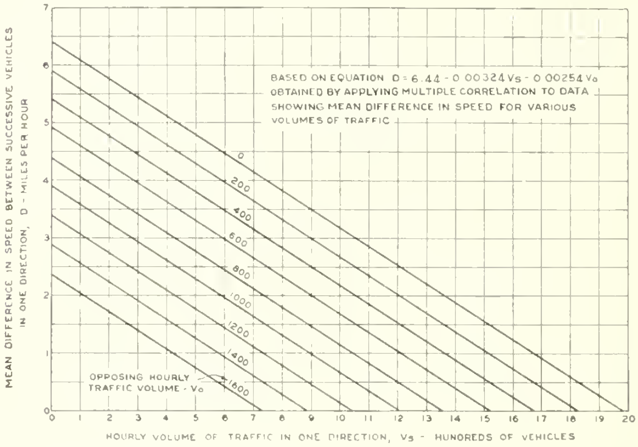

Figure 10.—Mean Difference in Speed Between Successive Vehicles With Various Volumes of Traffic. Section 3–J, 2-Lane Tangent.

By plotting the mean difference in speed for individual time period groups against the volume of traffic, a large scattering of points for each location was obtained as illustrated by figure 9. Some factor other than the volume of traffic in one direction evidently causes the large scattering of points.

A similar scattering occurs when total traffic volumes are used, but by applying multiple correlation to the data, using the mean difference in speed as the dependent variable and the volume of traffic in one direction together with the volume of traffic in the other direction as the two independent variables, the equation shown in figure 10 was developed. The series of lines obtained by solving the equation accounts for the wide scattering of points in figure 9. It is possible that a slightly curved series of lines would fit the points better and result in a slightly higher coefficient of correlation, but the coefficient of −0.812 indicates that the straight-line equation is highly significant within the range of values covered by the data.

To illustrate how figure 10 may be used in determining the relative interference between vehicles at various traffic volumes, assume that the volume of traffic traveling in one direction on this two-lane tangent is

200 vehicles per hour. If the opposing traffic is 700 vehicles per hour, the 200 vehicles will have the same freedom as a traffic volume of 500 per hour with an opposing traffic volume of 300 per hour.

MAXIMUM HOURLY CAPACITIES COMPUTED FOR VARIOUS HIGHWAYS

The maximum number of vehicles per hour that can travel over this two-lane tangent before all vehicles must start traveling at the same speed as the preceding vehicle and the drivers have no individual freedom, is 1,980 per hour with all traffic in one direction, and 1,100 in each direction with balanced traffic (see fig. 10). Should it be desired to limit the working capacity to the volume at which the freedom of the average vehicle would not be restricted more than 30 percent of the difference between the restriction with practically no other traffic on the road, and the restriction when all vehicles are required to travel at the same speed, the hourly traffic volume could not exceed 600 vehicles with substantially all traffic in one direction, or 330 vehicles in each direction.

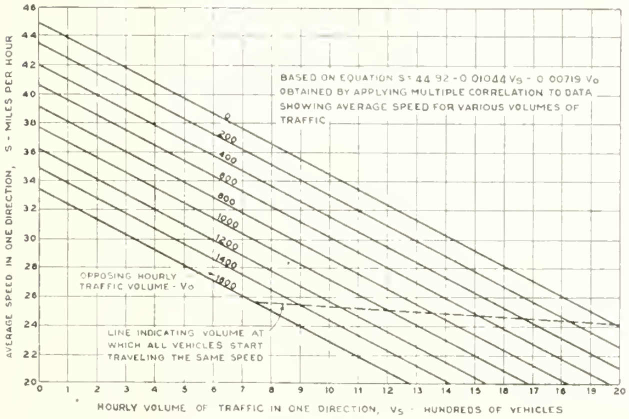

FIGURE 11.—AVERAGE SPEED WITH VARIOUS VOLUMES OF TRAFFIC. SECTION 3–J, 2-LANE TANGENT.

By correlating the speed of the vehicles with the volumes of traffic in the same and opposing directions, a coefficient of correlation of −0.877 was obtained for the equation shown in figure 11 and represented by the series of lines. It is obvious that the relationship will not hold true for values below the dashed line indicating the traffic volumes at which all vehicles must travel at the same speed according to figure 10. For values below this dashed line, one slow-moving vehicle could prevent all following vehicles from traveling faster than its speed. In determining practical working capacities, only traffic volumes lower than the values at which such a condition exists need be considered.

The same type of curves as those presented for this two-lane highway were constructed for a number of other sections of highway. To illustrate and compare a few of the results, figure 12 has been constructed using only the values representing an even distribution of traffic between the two directions. Results are shown for seven study sections, all tangents, including 2 two-lane, 1 three-lane, 2 four-lane undivided, and 2 four-lane divided highways.

The total maximum hourly capacity of one of the two-lane highways was 1,880 vehicles at a speed of 23 miles per hour. For the other, it was 2,200 vehicles at a speed of 25 miles per hour.

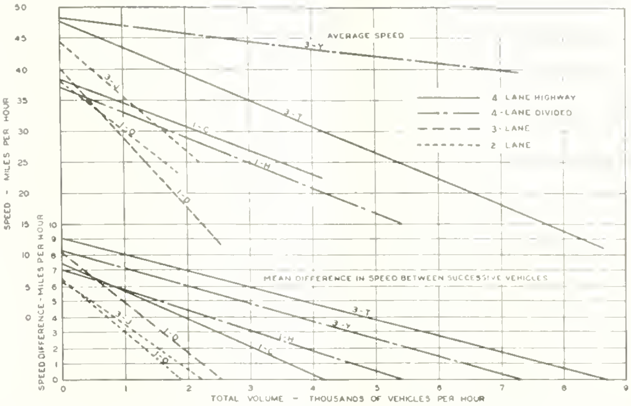

FIGURE 12.—SPEED AND MEAN DIFFERENCE IN SPEED FOR LEVEL TANGENTS WITH VARIOUS TOTAL VOLUMES EQUALLY DISTRIBUTED BETWEEN THE TWO DIRECTIONS.

The maximum hourly capacity of the three-lane highway was 2,540 vehicles at a speed of 12 miles per hour. One of the four-lane highways had a total maximum capacity of 4,150 vehicles per hour at a speed of 22 miles per hour, and the other a total maximum capacity of 8,600 at a speed of 11 miles per hour. One of the four-lane divided highways could accommodate a total maximum of 5,400 vehicles at a speed of 15 miles per hour, and the other 7,300 vehicles at a speed of 40 miles per hour. It should be remembered that these total maximum hourly capacities and corresponding speeds are for tangent sections, and that at the maximum volumes each driver must govern the speed of his vehicle entirely by the speed of the preceding vehicle, a condition that the average driver considers very unfavorable, especially for rural highways.

It must also be remembered that the values for these sections do not represent values for all highways having a corresponding number of lanes. These particular sections cannot be considered typical two-lane, three-lane, and four-lane highways, but were selected to illustrate that in determining the practical working capacity of a highway, consideration must be given to the speed at which the vehicles will be able to travel and the relative interference between vehicles that will be present. Thus, if it were desired to maintain an average speed of 35 miles per hour, the working capacity for the two-lane section 3–J would be 1,100 vehicles per hour, and for the four-lane section 3–T it would be 3,000 vehicles per hour. If it were also desired to limit the interference between vehicles to that present when there is a mean difference in speed of 4 miles per hour, the working capacity of section 3–J would be limited to 850 vehicles per hour. Only after analyzing data collected at many locations can generalizations be drawn for highways of various widths.

It is evident that practical working capacities are relative rather than absolute values, just as the safety factors used in structural design are relative rather than absolute values. The speed and interference factor selected for a highway in determining its practical working capacity must be governed by local conditions and the interference that the particular type of traffic carried by the highway will tolerate.

In designing highways for present or future traffic volumes, consideration must also be given to the individual traffic flow diagrams indicating the number of times certain hourly volumes will be exceeded and the percentage of the total vehicles traveling in each direction during such periods.

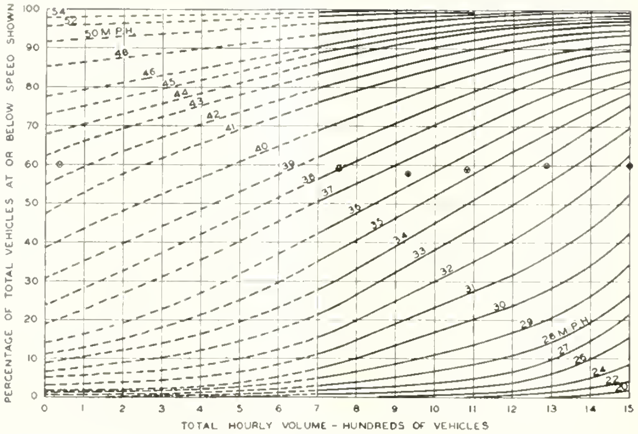

Figure 13.—Cumulative Percentage Distribution of Speeds in Relation to Volume of Traffic. Section 3-J, 2-Lane Tangent.

So far, this analysis has been limited to level tangent sections with long sight distances. The effect that grades, intersections, and various degrees of curvature have on the capacity must be determined before it will be possible to arrive at practical working capacities for highways of various designs.

Highway transportation plays such an important part in the life of everyone that it is surprising how little is known about the fundamentals of traffic behavior. Great care is taken in the construction of highways, yet comparatively little is known regarding the effectiveness with which the facilities provided fulfill traffic requirements. A number of figures illustrating a few of the fundamentals of traffic behavior have been constructed from the capacity study data. These should be useful to traffic and highway engineers.

REGARDLESS OF VOLUME, THE SAME PERCENTAGE OF VEHICLES WENT SLOWER THAN THE AVERAGE SPEED

Figure 13 was constructed by plotting the percentage of vehicles traveling at or below given speeds for various total volumes of traffic on a certain two-lane rural highway. The solid lines cover the range of hourly volumes observed during the study. In order to extend these lines to cover lower volumes, a frequency distribution of speeds for all individual vehicles spaced at least 2,000 feet behind the preceding vehicle and without interference from oncoming traffic was used to represent the distribution of speeds as the volume approached zero. As far as the design of the highway was concerned, all of these vehicles could have traveled 50 miles per hour. However, observations of these vehicles which represent low traffic volumes showed that 10 percent of the vehicles traveled less than 35 miles per hour and only 8 percent traveled faster than 50 miles per hour. With a traffic volume of 1,000 vehicles per hour, 53 percent of the vehicles traveled at or less than 35 miles per hour, and only 4 percent went faster than 50 miles per hour.

The circles in figure 13 represent the average speeds for various total hourly traffic volumes. Regardless of the total traffic volume on this highway, the percentage of the vehicles traveling slower than the average speed remained practically constant. Were the curves extended, at a total traffic volume of 2,200 vehicles per hour, the average vehicle speed would be approximately 25 miles per hour, which checks with the average speed previously determined for the total maximum capacity of 2,200 vehicles per hour at this same location.

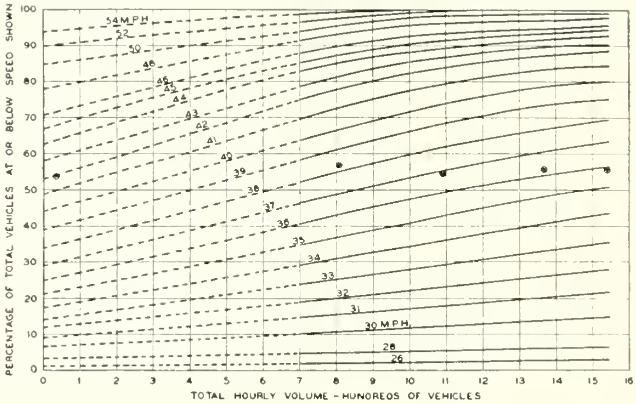

Figure 14.—Cumulative Percentage Distribution of Speeds in Relation to Volume of Traffic. Section 1-M, 3-Lane Tangent.

Figure 14 shows the same data for a three-lane tangent. The speeds at the low traffic volumes are slightly higher than for the two-lane tangent and change less with an increase in volume. On this highway the percentage of vehicles going slower than the average speed of all traffic also remains practically constant as the total traffic volume changes, but is approximately 55 percent rather than 60 percent.

The curves for a four-lane divided tangent shown in figure 15 and those for a two-lane tangent shown in figure 16 also reveal that a fairly constant percentage of the vehicles travel slower than the average speed, the percentage being 55 for the four-lane divided highway and about 70 for the two-lane highway.

Figure 17 shows the distribution of speeds on a four-lane undivided tangent on which the speed limit was 30 miles per hour. At the low traffic volumes about 60 percent of the vehicles exceeded the speed limit. The range of speeds at the low volumes was much less than for highways on which speed was not restricted. The frequency distribution curves for the speeds did not change to any marked extent until a traffic volume of 1,600 to 2,000 vehicles per hour in one direction was reached. Here again, it is seen that the percentage of vehicles traveling slower than the average speed did not vary to any marked degree as the volume changed. These figures and similar data for other locations indicate that although the percentage of vehicles traveling below the average speed varies for different highways, the percentage traveling below the average speed on any particular highway does not change an appreciable amount with a change in traffic volume.

The data recorded also made it possible to determine the frequency distribution of time spacings between successive vehicles traveling in the same direction for various volumes of traffic, as illustrated by figure 18. On this particular two-lane tangent when the volume in one direction was 300 vehicles per hour, 25 percent of the vehicles or 75 of the 300 vehicles passing each hour reached a given location on the highway less than 2 seconds after the preceding vehicle. Likewise, 37 percent, or 111 of the 300 vehicles, were less than 3 seconds behind the preceding vehicles. Twelve per cent, or 36 of the 300 vehicles, were, therefore, spaced between 2 and 3 seconds from the preceding vehicle.

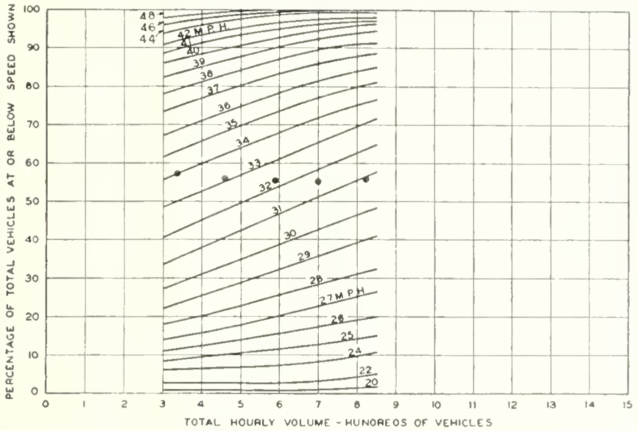

Figure 15.—Cumulative Percentage Distribution of Speeds in Relation to Volume of Traffic. Section 1-H, 4-Lane Divided Tangent.

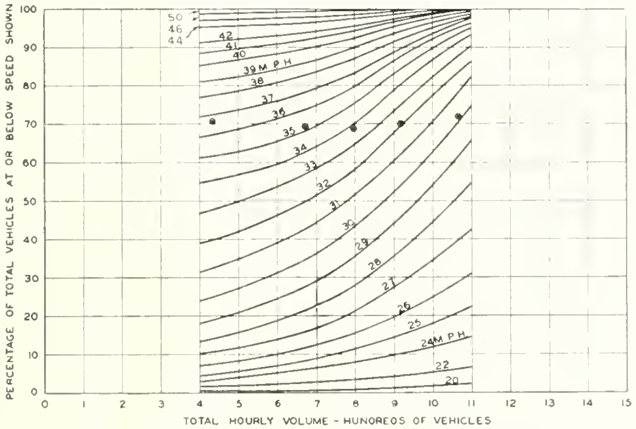

Figure 16.—Cumulative Percentage Distribution of Speeds in Relation to Volume of Traffic. Section 1-O, 2-Lane Tangent.

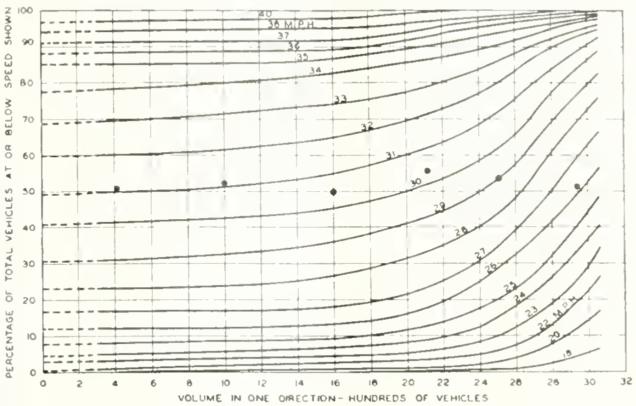

Figure 17.—Cumulative Percentage Distribution of Speeds in Relation to Volume of Traffic. Section 1-E, 4-Lane Undivided Tangent.

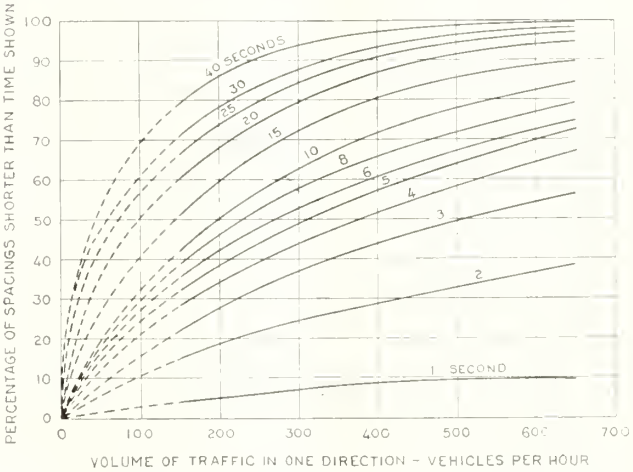

Figure 18.—Frequency Distribution of Time Spacings Between Successive Vehicles at Various Volumes of Traffic. Section 2-H, 2-Lane Tangent.

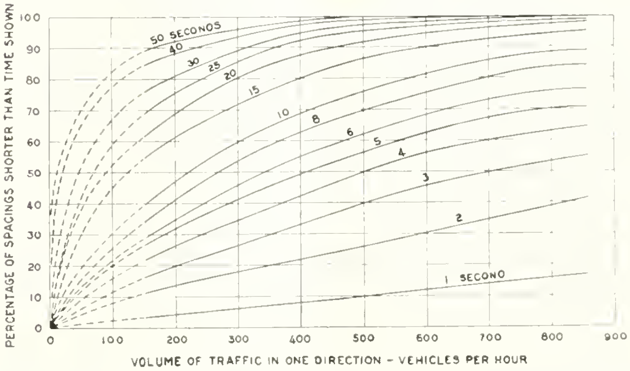

Figure 19.—Frequency Distribution of Time Spacings Between Successive Vehicles at Various Volumes of Traffic. Section 2-J, 3-Lane Tangent.

All time spacings between vehicles in one direction passing in 1 hour obviously total 3,600 seconds, and the average time spacing in seconds may be determined by dividing 3,600 by the hourly traffic volume in that direction. One would normally expect to find 50 percent of the time spacings shorter than the average spacing and 50 percent longer than the average spacing. At this location, when the volume was 200 vehicles per hour and the average spacing 18 seconds, 66 percent of the spacings were less than 18 seconds. At 300 vehicles per hour, 66 percent of the spacings were less than the average spacing of 12 seconds, and at 600 vehicles per hour, 72 percent of the spacings were less than the average spacing of 6 seconds. Regardless of the traffic volume, from 66 to 72 percent of the vehicles were at spacings shorter than the average spacing, and from 45 to 55 percent of the spacings were shorter than one-half of the average spacing.

The distribution of time spacings for corresponding volumes of traffic on the three-lane section illustrated by figure 19 was almost the same as for the two-lane section, although the average speeds were not the same. At a volume of 200 vehicles per hour, 66 percent of the spacings were shorter than the average spacing of 18 seconds, and at 600 vehicles per hour about 68 percent of the spacings were shorter than the average spacing of 6 seconds. Forty-five percent of the spacings were shorter than one-half of the average spacing, regardless of the traffic volume.

The curves shown in figure 20, representing the time spacings for another two-lane section, are nearly the same as those for the other two-lane section, and all except those for the very short time intervals are nearly the same as those for the three-lane highway. From 68 to 75 percent of the spacings were shorter than the average spacing and (for volumes up to 800 vehicles per hour in one direction) from 45 to 62 percent

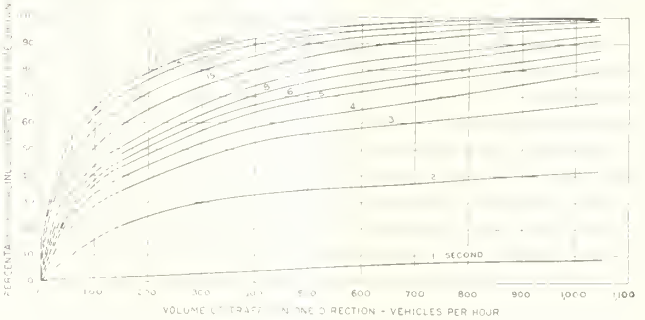

Figure 20.—Frequency Distribution of Time Spacings Between Successive Vehicles at Various Volumes of Traffic. Section 3–J, 2-Lane Tangent.

of the time intervals were shorter than one-half of the average time interval.

The most significant facts revealed by the study of the frequency distribution of the time spacings for a number of highway locations are:

1. As the volume of traffic increased on any particular highway there was a fairly uniform change in the frequency distribution of the time spacings rather than a sudden change at any particular volume of traffic.

2. Except for the very short time spacings, the frequency distribution of the spacings for any particular volume of traffic in one direction was practically the same on one highway as another regardless of the speed that the vehicles were traveling.

3. There was a very skewed distribution of the time spacings regardless of the volume or speed of traffic or type of highway. At any traffic volume, from two-thirds to three-fourths of the vehicles were at less than the average time spacing from the preceding vehicle.

TIME SPACING DATA USED TO DETERMINE OPPORTUNITIES TO PASS

A study of the distribution of distance spacings between successive vehicles showed the same relative distribution as the time spacing, but they varied with the average vehicle speed as well as with the volume of traffic. That is, a highway accommodating 300 vehicles per hour at a speed of 30 miles per hour would have the same percentage of the spacings exceeding 500 feet as the percentage that exceeded 750 feet on a highway accommodating 300 vehicles per hour at a speed of 45 miles per hour.

Traffic cannot move freely on a highway unless it is possible for faster moving vehicles to overtake and pass slow moving vehicles. On two-lane highways passing is not possible even on tangent sections except during periods when the left lane is not occupied by oncoming traffic. With the oncoming traffic moving at the same speed as the passing vehicle, the time spacing between succeeding vehicles in the opposing lane of traffic must be at least twice as great as the time required by the passing vehicles to perform the passing maneuver.

Characteristic patterns of time spacing distributions as illustrated in figure 20 can be used to determine the number of times per hour that any given time spacing between succeeding vehicles in the opposing lane will be exceeded. The percentage of the total time that a vehicle is opposite a time spacing in the opposing traffic which exceeds a certain magnitude may also be determined from the data. As an example, assume that a car is traveling on a two-lane highway with 400 vehicles per hour in the opposing lane of traffic. Out of every 400 vehicles that the car meets, 80 vehicles or 20 percent of the total vehicles will be 15 seconds or more behind the vehicle they are following. The 80 spaces total 2,448 seconds, so for 68 percent of each hour the car is opposite spaces equal to or exceeding 15 seconds. The dashed lines on figure 21 show the percentage of the total time that the car will be opposite spaces equal to or in excess of 15, 20, 25, and 30 seconds for volumes as high as 900 vehicles per hour in the opposing lane. These values will be the same regardless of the speed the car travels and will not vary appreciably for different average speeds of traffic in the opposing lane.

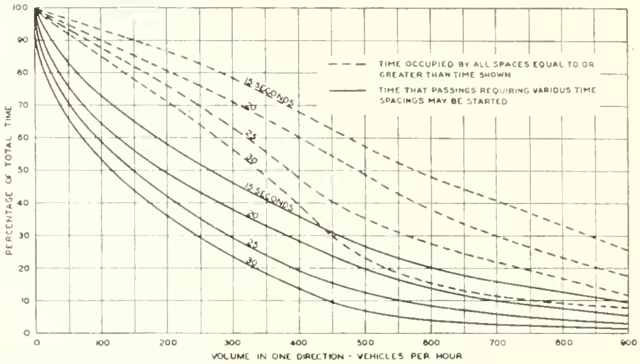

Figure 21.—Percentage of Total Time Occupied by Various Time Spacings Between Vehicles and Percentage of Total Time That Passings Requiring Various Time Spacings May Be Started. Section 3–J, 2-Lane Tangent.

A passing requiring a time interval of 15 seconds in the opposing traffic cannot be started after a portion of the 15-second interval has elapsed. It is only while the time interval is in excess of 15 seconds that such a passing may be started. To find the percentage of the total time that a passing may be started, it is necessary to subtract the required passing time interval from each of the intervals in excess of the minimum spacing. Thus from the total of 2,448 seconds occupied by the 80 spaces equal to or in excess of 15 seconds, 80 × 15 or 1,200 seconds must be subtracted leaving 1,248 seconds, or about 35 percent of the time, that such a passing may be started.

The solid lines of figure 21 indicate the percentage of the total time that passings requiring time intervals of 15, 20, 25, and 30 seconds in the opposing traffic may be started. Until the results of the passing studies recently conducted are analyzed, the exact time intervals required for various passing operations will not be known, but from available data it is believed that the intervals shown will include a large majority of the passings that occur on two-lane highways.

There are a number of ways in which this information may be used: First, assume that a fast-moving vehicle, traveling on a long tangent, is overtaking a slow-moving vehicle traveling in the same direction, and that a minimum interval of 15 seconds between vehicles in the opposing lane will be required to allow the fast-moving vehicle to pass. When the opposing traffic is 150 vehicles per hour, the chances are that 65 percent of the time, or two out of every three times, the passing maneuver can be made without first slowing down and waiting for an opportunity to pass. With an opposing volume of 600 vehicles per hour the chances are reduced to 20 percent or 1 out of every 5 times.

For the second illustration, assume that the fast-moving vehicle has been required to slow down to the

(Continued on page 240)

COMPARISON OF METHODS FOR DETERMINING THE HILL CLIMBING ABILITY OF TRUCKS

Reported by CARL C. SAAL, Assistant Highway Engineer, Bureau of Public Roads

ONE of the most serious problems confronting highway engineers today is that of increasing the traffic capacity of the more congested two-lane highways. The simple solution to this problem—the construction of additional traffic lanes—is prohibitive in cost for application to any but a limited mileage of the most heavily traveled roads. Alternative methods include the increasing of sight distances to provide more frequent opportunity for faster moving vehicles to pass slow vehicles, and the speeding up of slow-moving vehicles.

Particularly on hills do slow moving vehicles restrict traffic flow, with consequent delay, inconvenience, and hazard.

Commercial vehicles especially are frequently loaded so heavily that with the power available their speeds are so reduced on the steeper grades that long lines of vehicles accummulate behind them. Under such conditions, drivers become impatient and attempt passing maneuvers their better judgment would warn them against.

Such remedies as increasing sight distances or, as has been suggested, building additional lanes on hills, are localized in their character. A more comprehensive solution of the broad problem lies in measures to speed up the slower vehicles, either by reducing the steeper grades or by increasing the engine power of the slower vehicles. The State-wide highway planning surveys will reveal the number and lengths of excessive grades. From these figures, estimates may be made of the cost of reducing the steepest of the grades on the more heavily traveled roads to some more reasonable slope.

If the steepest gradients were, for example, 9 percent, it is possible that at no great expense all such grades could be reduced to 8 percent. At a greater cost, the more numerous 8-percent grades could be reduced to 7 percent, and so on, each successive step involving mileage increasing in a geometric ratio until some point would be reached beyond which it would be hopeless with available funds to attempt further reductions. The grade represented by this point would then be one over which speeds within the desired range must be maintained. The lower limit of this speed range is one at which all vehicles may travel without undue restriction of power or load, yet not so low that the drivers of following vehicles will by their impatience become reckless in their actions.

That the problem is becoming critical is evidenced by the numerous proposals advanced regarding the regulation of the performance of motor trucks. To forestall the imposition of hasty or ill-advised regulation, and to facilitate an intelligent evaluation of the relative economies of reducing gradients and of increasing the power of trucks, are the purposes of an exhaustive study of motor trucks now in progress. In cooperation with a number of truck manufacturers, the Quartermaster Corps of the United States Army, and the National Bureau of Standards, the Bureau of Public Roads is now determining the hill-climbing ability of some 30 new trucks, covering the range of sizes and makes most generally encountered in the eastern portion of the country.[1]

These studies will determine the maximum performances of new vehicles in the best of condition, operating under the heaviest loads they may be expected to carry. That throughout the life of these vehicles their performance will remain at the original level is unreasonable to expect; accordingly similar studies must be conducted on used vehicles in various stages of wear. Finally, any vehicle, new or used, may perform much less satisfactorily under ordinary driving in actual use than when under test. The complete analysis of performance will then require a determination of the best performance, the loss of ability through wear and age, and the factor to apply to compute the normally anticipated performance from the maximum performance for the vehicle condition.

ACTUAL GRADE TESTS MOST ACCURATE AND COMPLETELY ADEQUATE METHOD

Data collected in the State-wide highway planning surveys will show the types of vehicles and loads they carry on any given highway. Use of these data in conjunction with information on the road profile and the results of the ability tests will reveal within very close limits the effect of any performance regulation that might be enacted. By judicious correlation between grade reduction and regulation of the loads and power of vehicles, reasonable freedom of movement may be effected with the greatest economy.

A number of methods are being used to determine the hill-climbing abilities of new motortrucks. They are: (1) Actual grade tests made by applying various loads to vehicles and observing the speeds that they can maintain on a series of known grades, (2) theoretical hill-climbing ability computed from engine-torque and power curves, (3) acceleration tests by which the drawbar effort available at various road speeds over the entire useful speed range of each gear is determined, and (4) drawbar dynamometer tests which measure the drawbar pull available over the entire useful speed range of each transmission gear.

The actual grade tests are the most satisfactory in that no question can be raised as to their adequacy. These tests are laborious and expensive, and thus would not be practical where it is necessary to test a large number of trucks in a short period of time. One of the purposes of the study now being conducted is to develop a method by which accurate results can be obtained more quickly and economically. The results of the grade tests will be used to evaluate the results obtained by various other methods.

This report discusses the results obtained from tests made by the various methods on one truck designated as truck A operating in third gear on five grades. The grade ability, as determined by the grade tests, is ex-

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[1] Facilities used in analyzing the data are provided by the Johns Hopkins University.

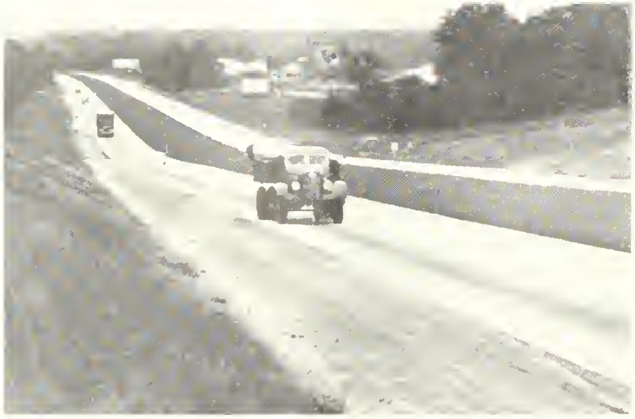

Figure 1.—Truck Climbing Grade in Grade Ability Tests.

pressed in terms of the gross vehicle weight that the truck can pull up the particular grade at a given speed. The results obtained by the other methods are converted to the same units. A comparison is then made between the results of the grade tests and those of each of the other methods.

Figure 3.—Chronograph Tape Record Being Analyzed in the Office

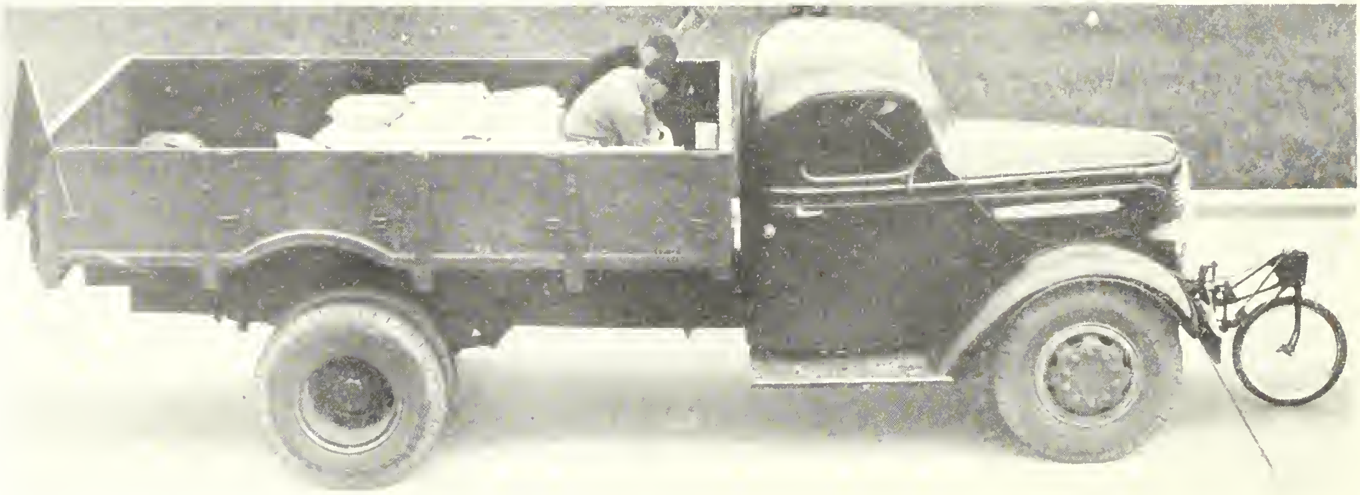

Figure 2.—Truck Equipped with Chronograph and Bicycle Wheel as Used in Grade Ability Determinations.

Grade tests.—In the grade tests known loads are applied to the truck and the maximum speed that it can maintain on known grades with each load is determined. The maximum gross vehicle weight that the truck can pull up the grade at a constant speed in a given gear is determined by trial, using the performance indicated by an ability formula as a guide. Starting with the maximum weight that can be hauled in a given gear, the load is decreased by 1,000-pound decrements and the maximum sustained speed for each load determined. The gross vehicle weight is decreased until the road speed observed corresponds to an engine speed that approximates the maximum recommended by the manufacturer for safe operation of the engine. As soon as the tests in one gear are completed, they are repeated in the gear with the next higher gear ratio. Figure 1 shows a truck climbing one of the test grades.

Several test runs on the grade are required to determine the maximum sustained speed for each load. The test truck approaches the grade at a speed estimated to be the speed that can be sustained over the entire length of the grade. All test runs are made with full throttle. An observer in the cab of the truck records the speed indicated by the truck speedometer at the start and finish of the run, and determines whether a sustained speed is reached. If the vehicle accelerates or decelerates on the first test run, the grade is entered on the next run at the speed that was recorded when the truck left the test course on the preceding run. After a constant climbing speed is observed on one run, it is verified by a check run. Generally the grade is reentered at speeds first above and then below that finally determined as the sustained speed to insure that it has been correctly determined.

The speedometer is used only as a guide in the field work. The actual speed maintained on the grade is measured by means of a time-distance recorder. The recording unit is a chronograph with three magnetic recording styles. Two brushes riding on a two-point cam, mounted on the axle of a bicycle wheel attached to the truck bumper, are wired in series with one of the styles, and cause each half revolution of the wheel to be recorded. A clock with six contacts on the second hand shaft wired in series with another of the styles furnishes a time record at 10-second intervals. A telegraph key, wired in series with a third style, is used to

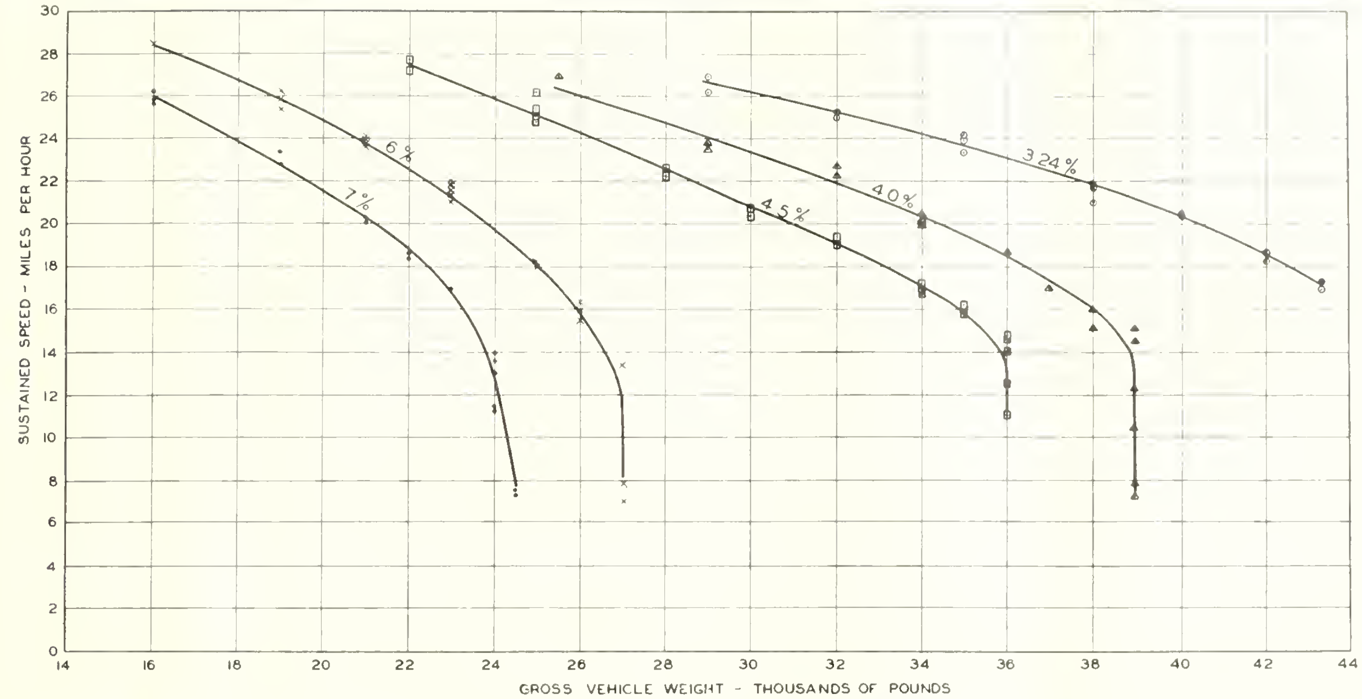

FIGURE 4.—HILL CLIMBING ABILITY OF TRUCK A OPERATING IN THIRD TRANSMISSION GEAR, BY ACTUAL GRADE TESTS.

mark the beginning and the end of the test course. Figure 2 shows a truck with the bicycle wheel attached to the front bumper.

Approximately instantaneous speeds are computed at short intervals by scaling the time required for a given number of revolutions of the bicycle wheel. The revolutions per second are computed and multiplied by the circumference of the bicycle wheel in feet to give the speed in feet per second, which in turn is converted to speed in miles per hour. Figure 3 shows a chronograph tape record being analyzed. From the record of instantaneous speeds it is possible to determine whether the vehicle is accelerating, decelerating, or maintaining a uniform speed at any desired time. Having thus determined the distance over which a uniform speed was maintained, the average speed for that entire distance is computed to give results with an error of less than 1 percent.

THEORETICAL PERFORMANCE CLOSELY CHECKED ACTUAL PERFORMANCE

The results obtained from grade tests on truck A operating in third gear on grades 3.24, 4, 4.5, 6, and 7 percent are shown in figure 4. The gross vehicle weights in thousands of pounds are plotted as abscissae and the speed in miles per hour as ordinates. The results shown here are the base data to which the results obtained from the other methods are compared.

Theoretical performance.—The theoretical performance of a vehicle is computed by reducing the engine torque through the transmission gears, axle gears, and driving wheels to rim pull or tractive effort. The tractive effort is the force produced, at the tire surface of the driving wheels, that is available to act against the resistances that oppose the motion of the vehicle.

The performance formula is derived by equating the tractive effort to the rolling resistance plus grade resistance. The tractive effort is obtained by dividing the torque at the driving axle by the rolling radius The torque produced at the driving axle for a given engine speed is the product of the engine torque, the total gear reduction, and the over-all efficiency. The grade resistance is the component of the gross vehicle weight along the grade and is the product of the tangent of the grade and the gross weight for all small grades, since the sine and the tangent of such angles can be considered equal without introducing substantial error. The rolling resistance is the product of the coefficient of rolling resistance, and the gross vehicle weight. The following formula then results: Tractive effort is equal to grade resistance plus rolling resistance.

$$\frac{T\times E\times R}{r}=(W\times f)+(W\times G)$$

or

$$W=\frac{T\times E\times R}{r\ (f+G)}$$

where T=Torque at a given engine speed, in pound-inches.
E=Over-all efficiency of vehicle.
R=Total gear reduction.
r=Rolling radius, in inches.
W=Gross vehicle weight, in pounds.
f=Coefficient of rolling resistance, in pounds per pound of weight.
G=Grade, in feet of rise per foot.

The torque as determined from engine performance curves for any given engine speed can be substituted in the above formula. The engine speed can be converted to road speed by reducing it by the total gear reduction and multiplying by the circumference of the driving wheels. In this manner the gross vehicle weight that can be pulled up a given grade at a given road speed can be determined.

The comparison between the hill-climbing ability indicated by the performance formula and that determined by actual grade tests is shown in figure 5. Again gross vehicle weight in thousands of pounds is plotted

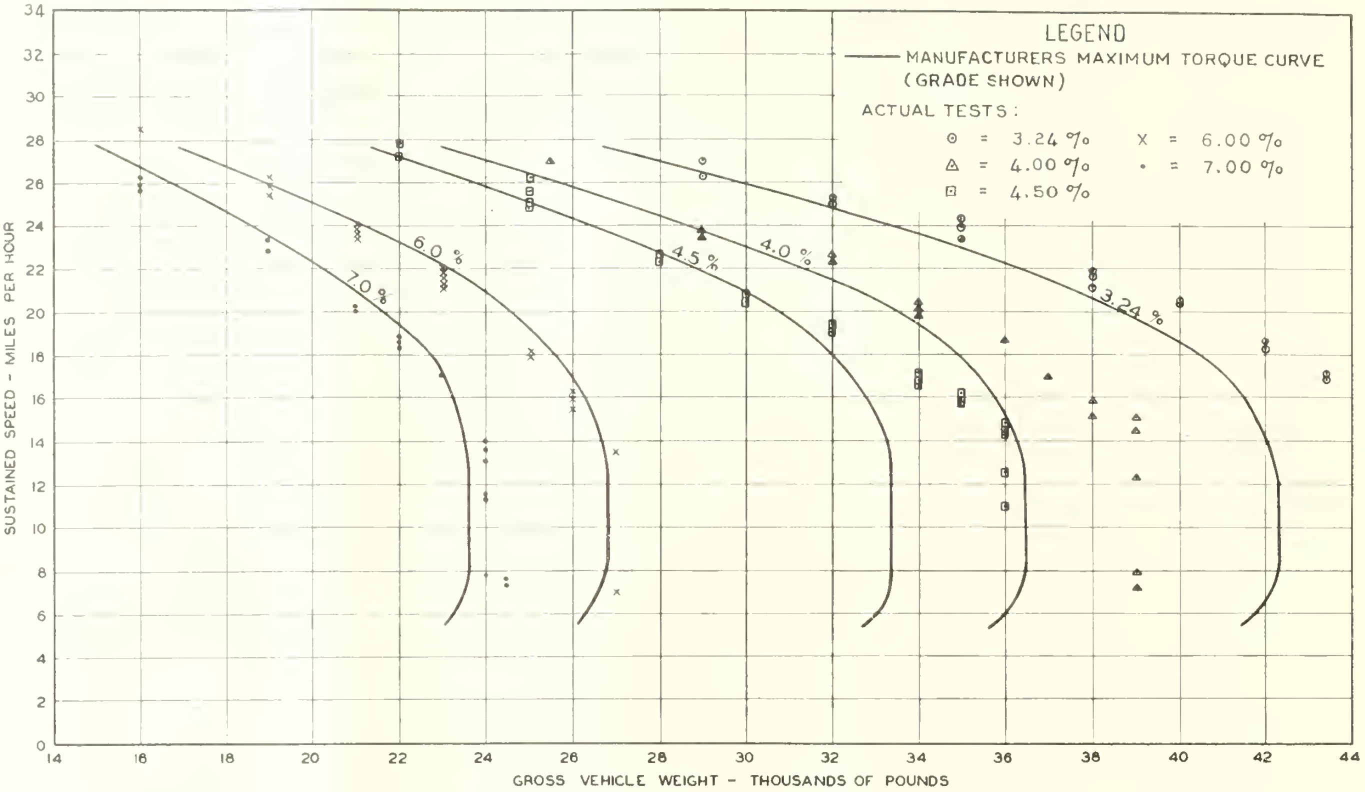

Figure 5.—Comparison of Hill Climbing Ability as Determined by Actual Grade Tests and by Performance Formula, for Truck A Operating in Third Gear.

against speed in miles per hour for each of the five grades. The plotted points indicate the actual performance, and the solid curves the theoretical performance.

An efficiency factor of 85 percent, and a coefficient of rolling resistance of 15 pounds per thousand, are assumed for computing theoretical performance. These values are the ones most commonly used. The engine torque is taken from curves furnished by the manufacturer. The rolling or loaded radius given by tire manuals for the size of tires on truck A is used.

The accuracy of the engine torque curves and of the assumptions made concerning efficiency and rolling resistence is indicated by the variation of the results obtained by the two methods. The results check remarkably closely except over the peak torque range. The rolling resistance as determined by deceleration tests on the level indicate that the coefficient of rolling resistance for truck A, operating at the road speeds at which peak torque is developed, is approximately 10 pounds per thousand. The difference between 15 pounds per thousand used here and 10 pounds per thousand is equivalent to ½ percent of grade. It will be noticed in figure 5 that results for the 4 and 4.5 percent grades verify this.

Dynamometer tests are being made on the engine of each truck after the grade tests have been completed. The results of these tests, together with the actual performance and the coefficient of rolling resistance measured by deceleration tests, will be used later to determine the true efficiency factor, and with these more accurate factors, it is believed that the performance can be computed with a fairly high degree of accuracy.

Acceleration and deceleration tests.—These tests are made on a level section of road. The truck is accelerated in each transmission gear at full throttle, starting at the slowest speed at which the engine will operate smoothly and continuing to the maximum recommended engine speeds. The drawbar pull or force produced at the drive wheels of the truck for any given road speed throughout the useful speed range of any gear is a function of the acceleration at that road speed and the mass of the truck.

SAMPLE COMPUTATIONS GIVEN FOR DRAWBAR PULL AND COEFFICIENT OF ROLLING RESISTANCE

Deceleration tests are made on the level by first attaining a desired speed and then permitting the truck to coast with transmission in neutral. The deceleration measured at a given road speed is proportional to the force that opposes the motion of the vehicle. This force or tractive resistance is composed principally of the friction between the tires and the road surface, and air resistance.

A time-distance record of each acceleration and deceleration run is obtained with the same time-distance recorder that was used in the grade tests. The time-distance record obtained is divided into two-second intervals, and instantaneous speeds are computed at each time interval. Time-speed curves are plotted whose abscissae are time in seconds and ordinates are speed in miles per hour. Thus the slope of the time-speed curve at any point is the acceleration or deceleration of the test truck. The acceleration or deceleration in miles per hour per second is determined at various road speeds and plotted against speed in miles per hour. The values of acceleration and deceleration as shown by these curves are used to compute the drawbar pull and the tractive resistance in the following manner:

Since the tractive resistance is a function of the deceleration and the mass of the vehicle, the following relation is true:

$$R=-a\ (M+K_0)\ldots\ldots\ldots\ldots\ldots\ldots\qquad(1)$$

where R=tractive resistance in pounds.

$-a$=deceleration, in feet per second per second.

K_0=mass equivalent constant for vehicle coasting in neutral.

M=mass of vehicle $\left(\frac{w}{g}\right)$.

The drawbar pull is proportional to the acceleration and the mass of the vehicle and the following relation is derived:

$$P=a\ (M+K_x)\ldots\ldots\ldots\ldots\ldots\ldots\qquad(2)$$

where P=drawbar pull, in pounds.

a=acceleration of vehicle, in feet per second per second.

M=mass of vehicle $\left(\frac{w}{g}\right)$.

K_x=mass equivalent constant for vehicle operating in a given transmission gear.

Rotating parts of a vehicle store up energy when the vehicle is accelerating. Since in every ordinary motor vehicle the speed ratio of these parts to road speed is fixed, the magnitude of this stored energy is in constant relation to the mass of the vehicle. This mass equivalent constant is determined with the engine operating in each transmission gear and for the vehicle coasting in neutral.

The tractive resistance is computed at 2-mile-per-hour intervals. This force is divided by the weight of the test truck in thousands of pounds to obtain the coefficient of tractive resistance or rolling resistance. Figure 6 shows the coefficient of rolling resistance plotted against speed in miles per hour. A typical point is determined as follows: At 20 miles per hour the deceleration is −0.258 feet per second per second. The weight of the vehicle (23,000 pounds) divided by the acceleration of gravity (32.2 feet per second per second) equals the mass of the vehicle (714). The mass equivalent constant for the vehicle coasting in neutral is 82. The tractive resistance, computed by equation (1), is found to be 205.4 pounds. This force of 205.4 pounds is divided by the weight of the vehicle in thousands of pounds (23) to give the coefficient of rolling resistance. The coefficient is 8.9 pounds per thousand and is plotted opposite 20 miles per hour on figure 6.

The drawbar pull is also computed at 2-mile-per-hour intervals over the useful speed range of each transmission gear. For truck A operating in third gear the results are shown in figure 7, in which drawbar pull in pounds is plotted against speed in miles per hour. A typical point is determined as follows: The acceleration measured at 20 miles per hour is 1.788 feet per second per second. The mass of the vehicle (714) is the weight (23,000 pounds) divided by the acceleration of gravity (32.2 feet per second per second). The mass equivalent constant for truck A operating in third gear is 132. The drawbar pull is therefore 1,513 pounds as computed by equation (2). This force is plotted opposite 20 miles per hour in figure 7.

The total force produced at the tire surface of an accelerating vehicle at any given speed is equal to the drawbar pull plus the tractive resistance of the test vehicle. This force can be utilized to pull a certain gross vehicle weight up a given grade at the road speed for which the force is measured. The force is equated to the component of the gross vehicle weight along the grade and the tractive resistance of the vehicle on the grade. The gross vehicle weight is computed, using the following formula:

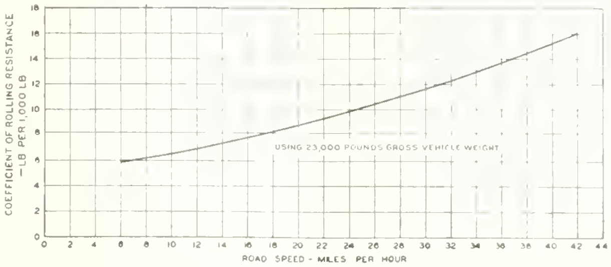

Figure 6.—Relation Between Coefficient of Rolling Resistance and Speed, for Truck A.

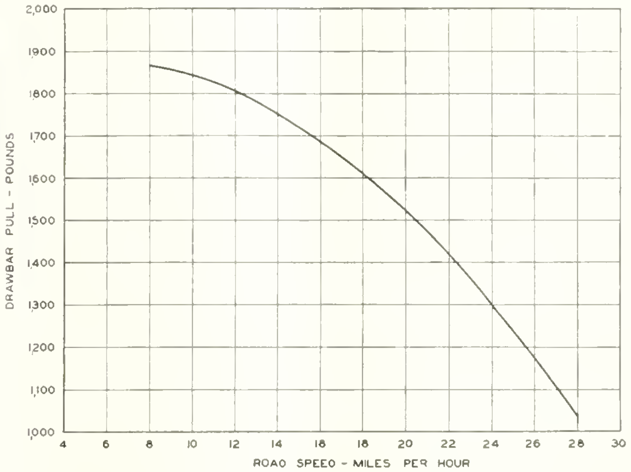

Figure 7.—Drawbar Pull Available at Various Road Speeds, for Truck A.

$$P+(w\times f)=(W\times G)+(W\times f)\ldots\ldots\ldots\ldots\qquad(3)$$

where

P=the force proportional to the acceleration of the vehicle.

$(w\times f)$ (weight of vehicle×coeficient of rolling resistance)=tractive resistance of accelerating vehicle at speed for which drawbar pull was measured.

$(W\times G)$ (gross vehicle weight×tangent of gradient)=grade resistance.

$(W\times f)$ (gross vehicle weight×coefficient of rolling resistance)=tractive resistance of vehicle on grade.

DYNAMOMETER TESTS QUICKLY PERFORMED AND RESULTS EASILY COMPUTED

The drawbar pull measured at various road speeds for truck A operating in third gear is converted to grade ability for the five grades. The results are shown in figure 8, in which gross vehicle weights are plotted against speeds in miles per hour. The solid lines indicate the results obtained by acceleration tests and the plotted points again show the results obtained by the actual grade tests. Typical points are determined as follows: The drawbar pull at 20 miles per hour is 1,513 pounds, as indicated in figure 7. The coefficient of rolling resistance at 20 miles per hour is 8.9 pounds per thousand, from figure 6. The tests were made with a gross vehicle weight of 23,000 pounds. For the test grades of 3.24, 4, 4.5, 6, and 7 percent the gross vehicle weights that can be carried on each grade at 20 miles per hour are 41,500, 35,100, 31,800, 24,900, and 21,800

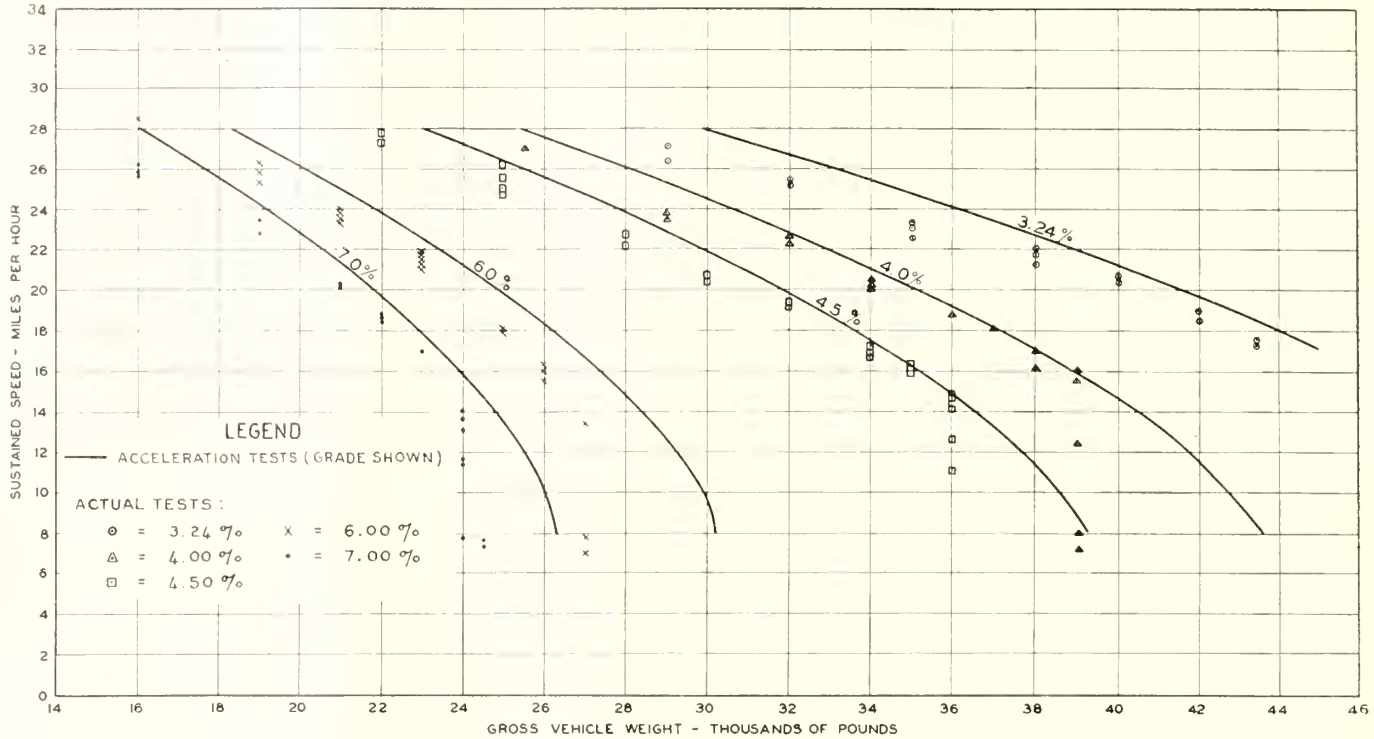

FIGURE 8.—COMPARISON OF HILL CLIMBING ABILITY AS DETERMINED BY ACTUAL GRADE TESTS AND ACCELERATION TESTS, FOR TRUCK A OPERATING IN THIRD GEAR.

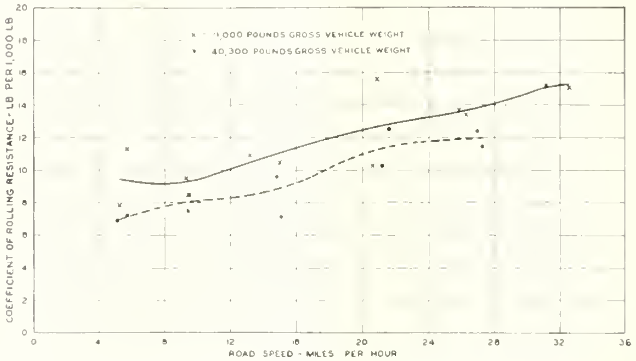

FIGURE 9.—RELATION BETWEEN COEFFICIENT OF ROLLING RESISTANCE AND ROAD SPEED AS MEASURED BY FIELD DYNAMOMETER, FOR TRUCK A.

pounds respectively. These weights are plotted opposite 20 miles per hour in figure 8.

Drawbar dynamometer.—A special dynamometer, called a "field" dynamometer, developed by the Ordnance Department of the United States Army, was made available for these studies. This dynamometer consists of a 10-ton truck on which two fire pumps are mounted. The pumps are connected to the main propeller shaft back of the transmission by means of a two-speed auxiliary transmission. When the dynamometer vehicle is towed by the test vehicle, the rear wheels of the truck actuate the pumps, forcing water from a tank through a pipe with an adjustable orifice and then back to the tank. By regulating the size of orifice the discharge pressure of the pumps can be increased or decreased, which in turn increases or decreases the torque required to turn the rear wheels. In this manner various loads can be applied to the vehicle pulling the dynamometer.

Drawbars are mounted on the front and rear of the dynamometer truck. Each drawbar is composed of a cylinder and piston, the former being held stationary relative to the dynamometer truck, while the latter is connected to the drawbar eye. Thus when the test vehicle tows the artificially loaded dynamometer truck, the pressure in the cylinder is recorded through an oil line and spring gage on a metallic chronograph tape. At the same time, marks from which road speed can be computed are recorded on the same chronograph tape to permit, by relatively easy means, the computation of the available drawbar pull at various road speeds. Resistance to traction is determined by towing the test vehicle behind the dynamometer truck. All drawbar-pull tests are conducted on a level course at full throttle, the artificial load and, correspondingly, the speed of the vehicles being regulated by an operator who varies the size of the orifice as indicated by an electric tachometer.

Figure 9 shows the coefficient of rolling resistance in pounds per thousand pounds plotted against speed in miles per hour. The drawbar pull obtained by means of the dynamometer is converted to hill-climbing ability in terms of gross vehicle weight and speed in the same manner as for the drawbar pull measured by the acceleration tests. The results for truck A operating in third gear are shown in figure 10. The solid curve indicates the results of the dynamometer tests, and the plotted points those of the actual grade tests.

The grade ability measured by the two methods varies by an amount that is approximately equivalent to a grade of one-half of 1 percent, the curve for calculated ability on a 4-percent grade coinciding very closely with the points showing the actual ability on a 4.5-percent grade. Although the results of the tests by this method do not compare too favorably with the actual grade results, it should not be concluded that the method is unsound. The field dynamometer was designed for use in testing heavy, slow-speed equipment such as tractors and tanks. The results obtained by

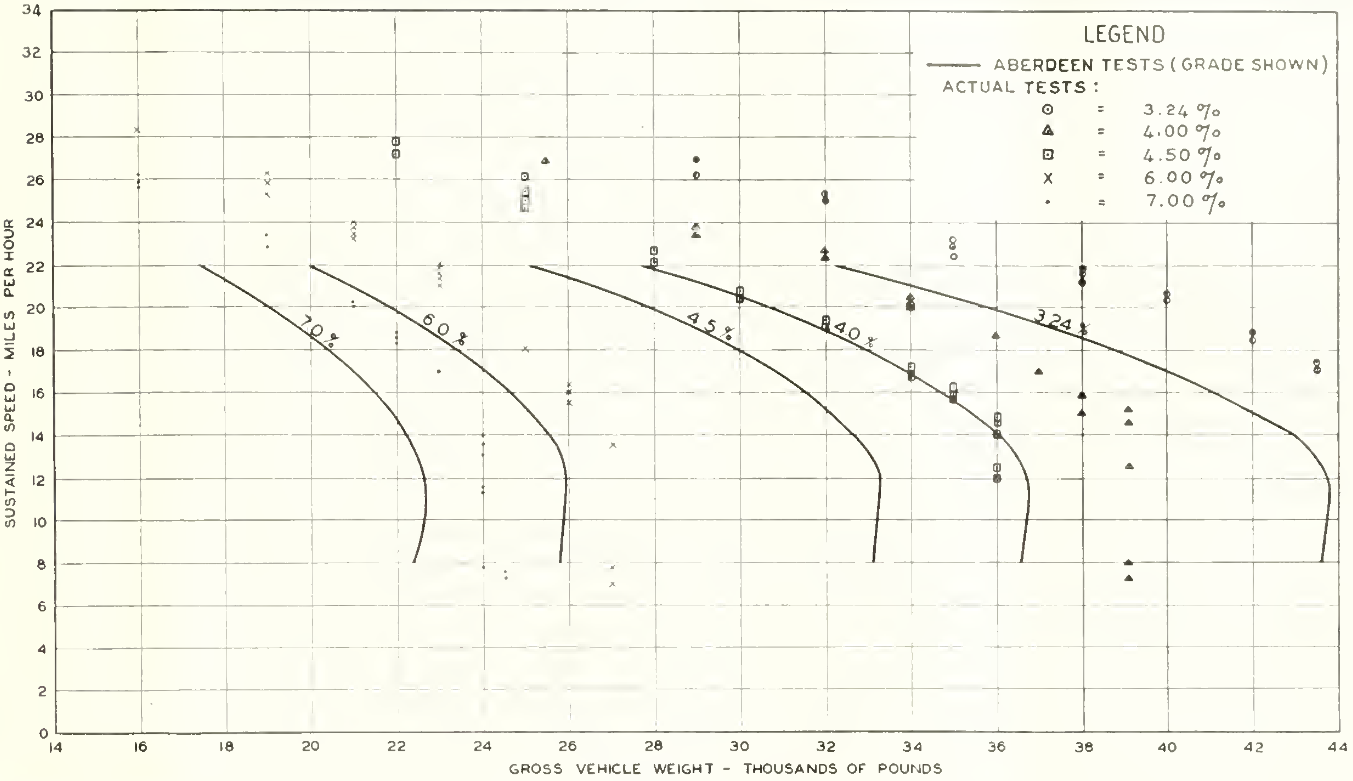

Figure 10.—Comparison of Hill Climbing Ability as Determined by Actual Grade Tests, and Aberdeen Proving Ground Drawbar Pull Tests, for Truck A Operating in Third Gear.

the two methods compared much more favorably when the vehicle was tested in the lower gears, although the speeds in those gears were well below normal road speeds. The differences were even more pronounced in towing the test vehicle to determine its tractive resistance, for the truck offered very little resistance and accordingly it was difficult to maintain a constant speed with the heavy dynamometer truck.

The method offers a distinct advantage in that relatively little computation is required to interpret the results, whereas involved calculations are required in analyzing the acceleration test data. Should a lighter dynamometer be available, it is likely that very accurate results could be easily and economically obtained.

The results discussed above have been confined to tests of one truck in one transmission gear. Similar comparisons have been made for each gear for each of the 18 units tested during the fall months of 1938. Additional new units will be tested in the spring months of 1939, after which attention will be directed to the testing of used vehicles.

It is anticipated that actual grade tests will be continued on all of the new vehicles, since they have been made available for exhaustive tests by various manufacturers who are cooperating in the study. Coincident with the determination of actual performance, other means of determining the performance such as those already described will be continued in anticipation of finally adopting the one that will provide data of sufficient accuracy most feasibly for the testing of large numbers of used vehicles.

CONCLUSIONS

Results thus far obtained, of which those presented here are reasonably typical, lead to the following conclusions:

1. Testing under actual load conditions over several grades will, of course, provide results of the greatest accuracy; however, this method is so laborious and expensive as to preclude its general usefulness.

2. The computation of grade ability from manufacturer's torque curves, provided reliable factors for overall efficiency and rolling resistance at various speeds are available, may be expected to yield reasonably accurate results.

3. The computation of grade ability from acceleration and deceleration tests will yield accurate results, especially in the more generally used range of engine speeds, and requires no data from the manufacturer. Complete tests require but a few hours in the field, but office computations are laborious. The method is, however, far cheaper than the actual grade tests, and the Bureau is now considering the development of instruments for a more precise determination of acceleration rates, in view of the probability of adopting this method for future work.

4. The towing dynamometer, or some similar device such as a chassis dynamometer, can undoubtedly produce accurate results quickly and in a usable form. Their high initial cost limits their use to operations requiring the testing of a large number of vehicles quickly.

(*Continued from page 232*)

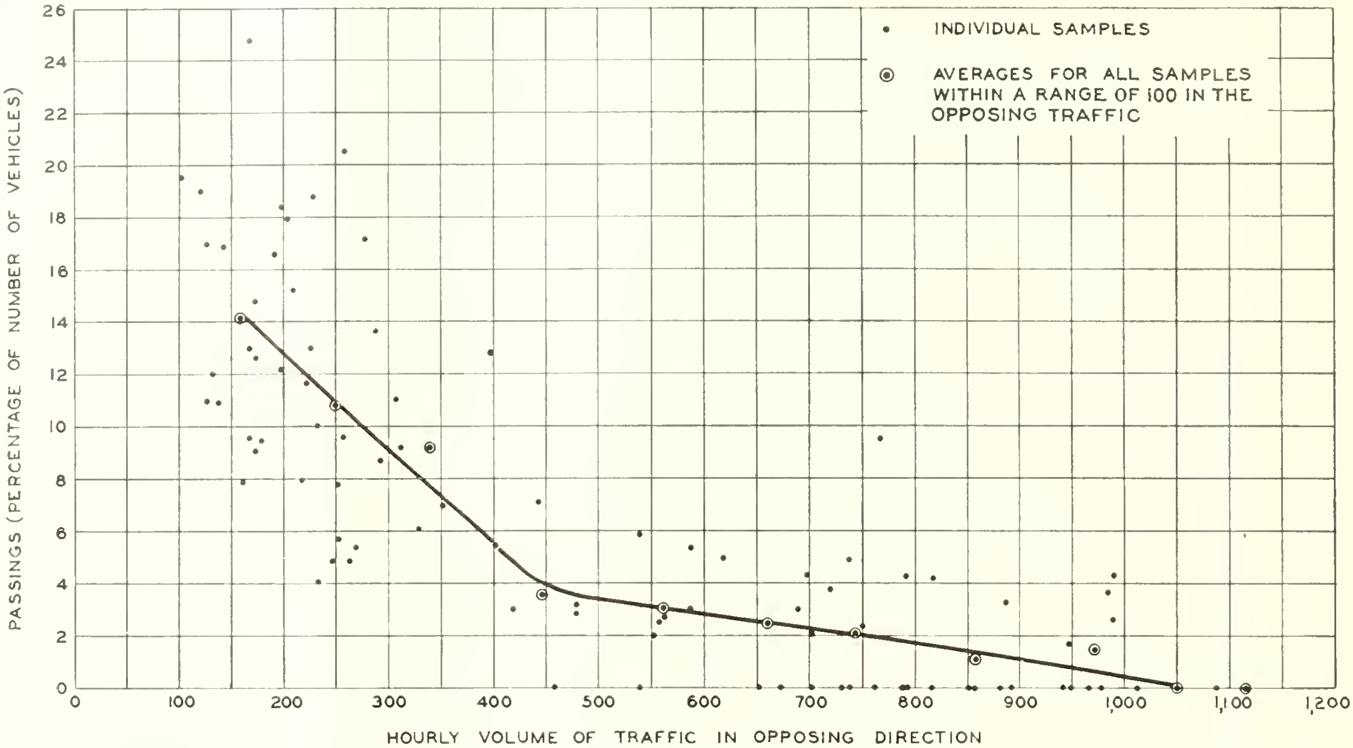

FIGURE 22.—PERCENTAGE OF VEHICLES PASSING IN ONE DIRECTION WITH VARIOUS VOLUMES OF TRAFFIC IN THE OPPOSING DIRECTION. SECTION 3-J, 2-LANE TANGENT.

same pace as the slow-moving vehicle and that a 25-second interval in the opposing traffic will therefore be required for the passing maneuver. With an opposing volume of 600 vehicles per hour, the vehicle desiring to pass will have to wait 6¼ times as long (on the average) as if the opposing volume had been 150 vehicles per hour.

For the third condition, assume that the alinement of the highway is such that passing cannot be accomplished with safety except on tangent sections just long enough to see that an interval of sufficient length exists in the oncoming traffic. When the required interval is 25 seconds and the opposing traffic is 150 vehicles per hour, the chances are that there will be no oncoming traffic on the passing section 50 percent of the time, but with an oncoming volume of 600 vehicles per hour, the chances are that the vehicle desiring to pass will have to stay behind the slow-moving vehicle for an average of 12 such passing sections before one opportunity to pass will present itself.

At a volume of 1,100 vehicles per hour in each direction, there is practically no opportunity to pass, even on a highway with unlimited sight distances. This was the maximum possible volume arrived at by using mean differences in speed as the congestion index.

It may also be seen from figure 22 that no passings were made on section 3-J when the traffic volume in the opposing lane exceeded 1,100 vehicles per hour.

CONCLUSIONS

The results that have thus far been obtained by analyzing a comparatively small portion of the data furnish sufficient proof to conclude that:

1. Neither maximum nor practical working capacities can be determined solely from the relation between average speed and volume.

2. Consideration must be given to both speed and the relative interference between vehicles in determining practical working capacities.

3. All highways of the same width or number of lanes will not have the same maximum possible capacities or the same practical working capacities.

4. Although there is a wide variation in the driving characteristics of individual vehicle operators, certain fundamental principles of traffic behavior can be developed that will be generally applicable. The results may be entirely different from those derived by assuming average conditions.

5. The various study sections for which data have been obtained do not cover a sufficient range of highway designs and alinement to obtain the effect of all the variable factors on practical working capacities. Further data are necessary, especially as to the time required for vehicles to pass one another under various conditions.

A NEW HANDBOOK ON TRANSITION CURVES FOR HIGHWAYS

A handbook entitled "Transition Curves for Highways," by Joseph Barnett, Senior Highway Design Engineer, has recently been issued by the Bureau of Public Roads. The Bureau recommends the general use of transitions to increase safety and ease of travel and to improve the appearance of highways. The book includes a set of tables with which the design and location of curves with transitions involve only simple calculations and it is practicable to project an initial alinement that includes transitions for horizontal curves without material delay or expense because of the transitions. This procedure has decided advantage over projecting the alinement as tangents with simple curves and later revising it to include transitions. Such revision frequently involves repeated trials with consequent expense and delay.

Sections of the handbook discuss speed in relation to highway design, design of curves with equal transitions by use of tables, design of curves with transitions as a general case, parallel transitions, transitions for compound curves, adjusting alinement of simple curves for transitions, widening pavements on curves, and right-of-way lines in relation to transitions. All tables needed in applying the methods described are included.

The handbook, in a durable binding, is available only by purchase from the Superintendent of Documents, Government Printing Office, Washington, D. C., at 60 cents a copy.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JANUARY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			* BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 5,498,126	$ 2,467,965	154.1	$ 8,404,552	$ 4,189,675	342.8	$ 1,261,780	$ 630,185	42.3	$ 3,532,108
Arizona	1,945,123	1,477,635	107.4	1,268,127	878,692	55.4	406,447	189,998	4.1	2,181,941
Arkansas	1,049,928	1,039,554	77.7	3,340,692	3,336,763	203.2	645,361	641,914	38.4	1,911,802
California	9,164,615	4,993,691	204.8	5,441,669	2,927,221	81.9	1,525,129	814,379	39.4	4,924,892
Colorado	2,489,482	1,347,520	90.5	2,804,813	1,478,357	93.8	1,531,850	851,780	31.5	2,861,883
Connecticut	934,030	455,835	8.9	684,518	337,455	7.9				2,149,026
Delaware	485,437	241,521	14.1	708,131	349,289	9.9	259,490	125,600	6.6	1,625,207
Florida	1,978,800	989,400	46.3	3,066,438	1,533,219	54.5	827,200	413,600	17.2	3,778,595
Georgia	4,512,812	2,205,528	225.4	4,817,250	2,408,625	227.9	1,127,860	563,930	83.4	7,717,488
Idaho	2,097,341	1,219,823	200.7	1,189,537	710,407	38.9	271,587	161,923	10.9	2,168,394
Illinois	10,983,560	5,457,228	299.7	7,228,026	3,610,414	151.8	3,318,011	1,658,960	76.8	5,016,862
Indiana	5,583,501	2,746,342	146.5	2,712,920	1,356,460	51.6	2,606,435	1,250,832	63.6	4,152,766
Iowa	7,372,868	3,437,619	238.9	4,901,212	2,072,883	156.9	526,096	119,000	34.8	2,605,932
Kansas	4,253,136	2,114,573	673.6	4,553,611	2,276,780	215.4	3,229,010	1,607,505	170.6	4,880,290
Kentucky	5,600,469	2,783,663	199.3	2,744,126	1,372,063	59.0	855,312	427,854	29.6	4,005,944
Louisiana	1,294,187	646,891	38.2	10,863,226	2,516,164	30.1	1,761,038	775,592	28.6	3,224,540
Maine	2,797,885	1,394,536	65.5	1,570,121	785,059	32.8	174,108	87,054	1.4	1,015,391
Maryland	1,085,456	542,728	17.1	2,516,978	1,233,351	40.7	726,570	363,000	10.2	2,362,285
Massachusetts	1,870,824	935,409	9.0	2,977,050	1,488,066	19.4	908,647	452,155	11.1	3,444,511
Michigan	7,698,130	3,716,055	163.7	4,024,208	2,011,452	117.7	1,348,325	672,430	32.3	3,829,757
Minnesota	4,760,858	2,296,147	289.3	6,086,869	3,020,665	279.3	1,153,760	575,975	61.2	4,629,540
Mississippi	2,049,958	940,417	82.8	10,043,462	3,857,052	435.4	1,723,590	666,270	73.3	3,525,814
Missouri	4,906,136	2,318,207	135.5	3,065,088	1,503,566	62.5	5,262,588	2,506,550	236.0	4,867,930
Montana	1,666,619	936,796	83.6	826,978	464,810	17.6	1,005,542	565,602	69.9	5,884,329
Nebraska	3,408,085	1,666,819	335.7	5,251,762	2,648,794	394.3	5,256,130	1,996,013	389.7	2,920,328
Nevada	1,443,091	1,236,906	168.8	1,301,341	1,122,550	51.0				2,015,222
New Hampshire	988,462	489,559	22.4	382,110	190,095	3.3	93,802	46,900	1.4	1,633,423
New Jersey	1,897,135	939,155	16.7	2,386,206	1,191,378	15.9	734,880	366,610	6.9	3,304,813
New Mexico	2,036,551	1,241,587	237.1	2,206,606	1,445,972	99.3	129,658	45,742	9.0	1,950,911
New York	14,153,092	6,947,158	253.2	9,751,777	4,830,992	158.6	3,347,230	1,527,398	60.2	5,033,757
North Carolina	6,425,599	3,088,829	254.8	5,110,619	2,554,152	342.3	838,540	395,000	41.0	3,727,947
North Dakota	3,372,957	3,241,828	261.5	500,901	290,805	57.5	69,522	37,236	6.8	5,091,159
Ohio	7,851,663	3,864,788	94.4	7,667,382	3,824,092	76.9	2,287,220	1,141,060	21.9	8,811,332
Oklahoma	5,118,083	2,691,081	226.1	3,493,243	1,802,200	79.6	1,692,800	892,645	51.6	4,491,195
Oregon	3,131,556	1,849,620	110.7	1,534,640	936,957	83.6	811,005	494,640	39.0	2,778,954
Pennsylvania	8,425,702	4,179,539	140.6	7,227,459	3,611,151	74.7	4,401,989	2,024,555	35.1	5,537,528
Rhode Island	1,179,290	589,645	16.4	372,212	186,106	3.5	307,320	153,660	3.2	1,394,638
South Carolina	4,543,984	1,999,748	247.1	3,362,381	1,508,776	94.4	287,891	137,700	11.1	2,493,886
South Dakota	1,974,839	1,106,433	245.7	4,674,676	2,585,190	441.5	339,070	187,490	27.8	4,157,378
Tennessee	5,183,970	2,576,497	168.0	2,973,769	1,487,606	49.6	1,503,040	751,520	44.7	5,413,859
Texas	11,548,473	5,711,218	757.0	12,852,190	6,347,476	554.4	4,063,009	1,946,379	284.6	9,017,839
Utah	1,055,963	740,744	112.0	2,139,223	1,517,650	73.4	109,720	78,320	4.2	1,605,990
Vermont	1,281,653	577,197	33.9	722,784	343,793	17.7	196,970	98,295	4.3	661,932
Virginia	5,918,394	2,957,016	204.9	2,956,476	1,476,203	89.1	876,084	435,937	11.0	2,160,077
Washington	3,977,752	2,080,870	99.5	2,751,911	1,441,059	28.3	369,697	193,800	7.9	2,082,024
West Virginia	1,784,387	1,287,765	65.4	1,296,682	649,266	26.5	916,770	454,535	26.7	2,977,222
Wisconsin	4,806,849	2,370,817	165.9	6,564,866	3,096,880	177.5	507,035	251,000	5.9	3,435,768
Wyoming	2,577,039	1,566,911	281.4	471,542	285,851	59.5	654,750	404,590	43.7	1,470,425
District of Columbia										487,500
Hawaii	834,497	414,174	18.0	822,330	403,270	9.3	22,380	8,600		1,692,677
Puerto Rico	53,980	26,990	2.1	1,428,309	710,910	29.0	292,606	144,945	5.8	839,030
TOTALS	197,052,328	102,147,977	8,111.9	186,042,999	92,207,662	5,877.1	62,564,854	30,336,658	2,316.7	173,484,041

* INCLUDES APPORTIONMENT FOR FISCAL YEAR 1940.

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF JANUARY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			* BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 197,900	$ 98,950	11.9	$ 699,535	$ 347,300	37.7	$ 145,300	$ 72,650	7.4	$ 844,346
Arizona	306,947	204,457	20.6	140,213	98,773	15.5	7,440	5,357		561,405
Arkansas				214,013	206,520	19.6	177,852	176,436	19.8	736,539
California	1,192,055	677,708	84.8	862,219	480,053	53.8	474,629	249,730	9.1	1,009,418
Colorado	812,169	443,250	51.1	393,760	218,125	19.3	163,150	75,059	3.9	456,322
Connecticut	69,450	34,705	1.3	41,584	20,632	.2	73,750	17,780	1.1	338,764
Delaware	18,950	9,475	4.0	53,394	26,697	8.8	47,240	23,135	9.9	260,693
Florida				294,238	146,661	1.6	225,517	112,400	11.2	616,044
Georgia	227,175	107,451	29.4	638,206	319,103	81.0	152,620	76,310	21.6	1,132,139
Idaho	451,521	204,347	46.9	166,441	87,870	11.9	25,544	15,263	1.1	350,359
Illinois	1,622,533	809,530	139.9	1,360,732	626,366	71.5	410,500	196,750	28.1	1,002,676
Indiana	599,857	255,400	64.8	679,900	322,150	62.0	694,048	337,152	62.8	691,623
Iowa										1,679,807
Kansas	53,582	26,791	8.6	209,772	104,885	16.3	175,226	87,613	32.3	1,487,558
Kentucky	779,275	240,584	100.9	712,452	186,076	28.3	779,443	229,948	88.6	435,839
Louisiana	64,069	31,900	6.9	548,671	245,325	43.1	525,449	231,310	43.7	431,113
Maine	362,500	181,250	23.3	262,662	126,214	12.5	18,700	9,350	1.2	142,777
Maryland				145,174	67,987	11.2	220,800	82,855	14.2	384,839
Massachusetts				139,441	69,604	2.4	222,970	110,905	4.9	672,214
Michigan	409,561	203,281	37.0	648,404	324,202	42.2	433,200	215,850	23.6	1,244,155
Minnesota	280,091	131,160	42.2	520,760	258,336	39.1	293,532	146,766	23.4	1,248,943
Mississippi				299,000	149,500	23.8	44,700	22,350	17.0	979,016
Missouri	320,539	152,877	43.1	326,360	145,320	28.3	763,280	335,900	103.8	879,539
Montana				27,601	15,525					1,324,368
Nebraska	464,760	230,241	81.4	409,692	198,615	66.1	537,358	265,209	102.0	610,393
Nevada	424,798	354,271	68.8	116,832	98,265	15.5	26,563	23,035	1.6	211,675
New Hampshire	245,058	121,630	6.0	60,759	29,708	2.3				168,662
New Jersey	123,040	61,520	2.4	199,860	91,195	2.6				718,263
New Mexico	563,070	343,413	36.9	642,199	391,668	41.0				310,985
New York	2,309,847	1,145,068	166.3	1,751,000	875,500	88.2	148,000	74,000	11.4	994,816
North Carolina	605,156	302,064	72.3	948,444	474,200	76.8	73,270	32,400	9.4	621,719
North Dakota	52,020	27,860	9.0	169,910	90,999	26.1	42,770	22,907	8.2	875,311
Ohio	156,560	78,280	3.8	100,970	57,260	1.8	473,200	236,600	25.2	1,996,979
Oklahoma	166,690	88,690	16.2	280,142	147,943	22.1	617,940	305,608	37.0	990,263
Oregon	421,378	247,170	56.3	59,454	36,322	5.0	32,248	18,940	2.2	698,524
Pennsylvania	1,699,993	823,463	120.7	1,751,101	857,769	95.8	635,950	317,975	33.1	750,639
Rhode Island	66,840	33,420	3.5	162,675	81,314	4.8	74,070	37,035	.9	109,247
South Carolina	404,550	174,382	43.5	806,307	336,469	88.1	228,170	93,300	16.7	273,761
South Dakota				11,300	6,250					1,058,050
Tennessee	221,720	110,860	11.0	555,244	204,722	28.8	199,040	93,050	4.7	962,828
Texas	2,441,362	1,143,243	349.0	1,804,074	779,023	175.1	896,775	416,884	111.0	1,719,683
Utah	446,508	233,685	41.1	236,967	127,060	18.5	166,230	79,254	16.1	279,081
Vermont	240,777	109,790	13.8	90,306	45,153	4.0	43,300	20,500	.5	107,278
Virginia	508,847	237,293	61.2	741,920	335,935	44.9	180,212	90,106	26.4	501,361
Washington	552,121	287,690	63.7	381,920	200,878	20.1	349,621	184,100	20.1	292,460
West Virginia	211,600	105,800	16.5	149,546	74,773	11.1	45,300	22,650	2.1	508,756
Wisconsin	490,676	230,918	23.0	695,644	341,190	31.7	83,660	41,190	1.4	961,346
Wyoming	416,146	254,582	59.0	285,102	176,169	11.9	121,478	75,041	10.4	266,758
District of Columbia										73,125
Hawaii				56,250	28,125	2.4				291,875
Puerto Rico	74,740	37,370	3.9	274,584	136,020	16.2				146,610
TOTALS	21,076,431	10,595,819	2,046.0	22,126,734	10,815,749	1,531.0	11,050,045	5,280,653	969.1	35,410,943

* INCLUDES APPORTIONMENT FOR FISCAL YEAR 1940.

PUBLICATIONS *of the* BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1938. 10 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . .	Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
Part 2 . . .	Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.
Part 3 . . .	Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.
Part 4 . . .	Official Inspection of Vehicles. 10 cents.
Part 5 . . .	Case Histories of Fatal Highway Accidents. 10 cents.
Part 6 . . .	The Accident-Prone Driver. 10 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP . .	The Results of Physical Tests of Road-Building Rock. 25 cents.
No. 191MP .	Roadside Improvement. 10 cents.
No. 272MP . .	Construction of Private Driveways. 10 cents.
No. 279MP . .	Bibliography on Highway Lighting. 5 cents.

Highway Accidents. 10 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Transition Curves for Highways. 60 cents.

DEPARTMENT BULLETINS

No. 1279D . .	Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.
No. 1486D . .	Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. 55T . . .	Highway Bridge Surveys. 20 cents.
No. 265T. . .	Electrical Equipment on Movable Bridges. 35 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

MISCELLANEOUS PUBLICATIONS

No. 296MP . .	Bibliography on Highway Safety.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y . .	Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I. Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in Public Roads, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF JANUARY 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					* BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 243,609	$ 243,410	6		4	$ 947,379	$ 945,524	9	1		$ 292,600	$ 292,600	6		3	$ 897,537
Arizona						9,452	9,452				236,674	225,087	3			521,135
Arkansas	253,258	252,101	7			468,537	468,257	8			101,133	101,133	2			1,281,009
California	669,417	669,417	2	2		1,338,720	1,338,095	8	1		562,382	561,890	2			1,859,663
Colorado	32,559	29,328	1		2	295,248	295,248	4			355,770	347,638	2		19	867,313
Connecticut						18,930	12,665									998,900
Delaware	39,000	39,000			13						47,420	47,420	2		10	504,830
Florida						445,510	445,510	3			75,900	75,900	1			1,161,858
Georgia						382,930	382,930	6			66,020	66,020	1		1	2,445,720
Idaho	174,973	174,800	4			280,682	249,386	4			131,898	120,500	1			439,963
Illinois	369,500	369,500	2		1	2,160,905	2,160,905	14	2	60	910,740	910,740	7		16	2,794,281
Indiana	688,715	599,689	3	5		875,513	848,613	3	1	90	152,200	152,200		1		1,328,146
Iowa	1,026,186	972,425	10	1	19	203,820	191,700	2	1	1	182,684	168,308	8		1	1,780,596
Kansas	453,985	453,880	7		4	671,850	671,850	10			511,160	511,160	5		9	1,442,577
Kentucky	145,000	145,000	1			344,297	344,297	4	1	31	471,324	471,324	6	3		1,210,270
Louisiana						233,220	226,478	3			657,724	623,090	11			1,038,359
Maine	48,590	48,590	2			332,396	332,396	3	2		69,630	69,630	1		1	375,778
Maryland						72,188	72,188	1								1,155,308
Massachusetts	54,710	54,710		1		220,486	220,290	1	1		386,385	385,425	1	3		1,661,149
Michigan	930,783	924,372	8	1	35	653,796	653,796	6	2	10	114,000	114,000				2,242,164
Minnesota	39,556	39,556		1	1	760,185	759,864	3	5		18,297	18,297			3	2,152,975
Mississippi	70,800	70,800	1			394,200	394,200	5			323,900	323,900	2			1,105,901
Missouri	296,674	295,552	4	1		111,370	111,370	1			1,027,890	1,027,890	4	3		2,186,625
Montana	355,586	350,704	4			634,520	634,520	6			235,773	235,773	3			364,726
Nebraska	150,374	150,374	4			729,314	729,314	15			757,668	757,668	18		33	474,743
Nevada	149,761	149,761		3	2	203,311	203,311	2			11,280	11,280			4	192,221
New Hampshire	70,205	69,765		1	2	87,856	87,797	5		1						433,688
New Jersey	116,891	111,665	1	1		229,856	229,856	1	1	2						2,018,350
New Mexico	168,984	168,984	4	1		118,994	118,994	3								730,470
New York	992,501	991,800	4	3	2	1,720,351	1,712,101	5	7		413,050	239,610		3	1	5,162,367
North Carolina	121,550	121,550	1	1	1	877,900	845,200	6	5		348,760	348,760	1	2	19	1,629,861
North Dakota	184,700	184,700		1		663,380	614,978	4	1							1,088,707
Ohio						515,870	515,870	6			403,610	377,610	3	1		4,169,541
Oklahoma						143,223	109,223	1			250,305	250,305	1		55	2,377,022
Oregon	308,391	307,742	1	2		384,601	250,243	1			129,997	129,997	2			521,579
Pennsylvania	213,129	197,923	2			1,039,656	827,757	2			931,522	926,878	3			4,908,593
Rhode Island						335,019	335,019	1	2		103,772	103,772		1		152,459
South Carolina	33,376	33,376		1		220,803	165,837	4	1		260,645	260,645	11		40	1,322,994
South Dakota	118,146	118,146	2		8	286,368	286,368	3	2	1	16,910	16,910	2		2	1,167,491
Tennessee						121,490	121,490		2		235,390	235,390	1		7	1,908,340
Texas	34,033	33,377	2			1,651,504	1,621,002	16	6		581,912	569,405	10			4,208,194
Utah	101,648	101,648	2		4	47,359	47,359	2			87,300	87,300			32	465,992
Vermont	215,052	209,695	6	1	5	29,964	29,964		1	1	23,000	23,000			7	315,293
Virginia	330,059	330,059	13	1		332,070	332,070	6	1		451,011	346,361	4	1	24	1,164,173
Washington	247,816	244,475	2	3	2	735,775	734,364	9	1		153,102	153,102	1		13	553,732
West Virginia	221,805	220,785	1		2	311,726	295,966	3		1	31,400	31,400	2		3	1,036,663
Wisconsin	221,661	221,493	3			1,186,812	1,145,988	11			4,917	4,917			1	1,587,185
Wyoming	164,200	164,200		2	4	10,150	10,150				201,010	121,590		1	7	512,924
District of Columbia						30,215	30,215		1							392,716
Hawaii						193,200	193,200	3	1							398,050
Puerto Rico						214,569	213,370	4								667,006
TOTALS	10,057,183	9,864,352	110	33	111	24,277,470	23,576,540	217	49	198	12,328,065	11,845,825	127	19	311	71,377,137

* INCLUDES APPORTIONMENT FOR FISCAL YEAR 1940.

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